# EEL CREEK STABILIZATION PROJECT

## HAMPTON, NH

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Natural Resources Conservation Service
United States Department of Agriculture

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# INTRODUCTION

Eel Creek and Meadow Pond are a natural salt marsh ecosystem. Eel Creek is the outlet for Meadow Pond and discharges into Hampton Harbor Salt Marsh. The creek runs under a bridge located on Winnacunnet Road. The bridge was replaced in 1996 after Hurricane Bob washed out the old structure.

The new structure was sized to allow adequate tidal water to flow into Meadow Pond. The old structure restricted the flow of tidal water into the pond. Proper tidal flushing is necessary to keep a salt marsh healthy. Since the old structure restricted tidal flushing, Meadow Pond and the surrounding marsh was turning into a fresh water ecosystem.

It was also determined that the new bridge would reduce the chance of flooding upstream of the bridge. The old structure impounded more water than the proposed new structure. Thus the water level in Meadow Pond for a 1% chance storm event (100-year storm) is lower with the new structure.

Since the replacement of the bridge, the Sandpiper Bay Condominium Association and the owner of the property located at 566 Winnacunnet Road (Figures 1 and 2) have reported stream bank erosion problems. The Natural Resources Conservation Service (NRCS) was asked to assist in the hydrologic and hydraulic analysis and determine possible corrective alternatives to stabilize the erosion along Eel Creek.

### Sizing the New Bridge

When the old conduit failed, NRCS was contacted to review the proposed bridge design to determine if it would allow adequate tidal flushing of Meadow Pond and not increase flooding problems. NRCS only had approximate dimensions of the conduit. Therefore, the old structure was modeled as if it were two 3.0 ft x 3.5 ft box culverts.

Hydrologic modeling was done to see what the water surface level (WSL) would be if a 1% chance storm event occurred while there was a high tide of 5.2 ft National Geodetic Vertical Datum (NGVD) in Meadow Pond and on the ocean side of the bridge. A WSL of 5.2 ft NGVD was used as the average WSL in the pond at high tide. A high tide of 5.2 ft NGVD would naturally occur in the pond if the bridge and the creek do not restrict flow.

Modeling showed the WSL would be 0.02 ft lower in Meadow Pond with the new structure than the old conduit for a 1% chance storm. Because the old structure was smaller, it conveyed less storm water than the new bridge and impounded more water in Meadow Pond at a higher elevation. Because more water was impounded, the WSL was higher.

## Aerial Photo of Site


Figure 1.

## Topographic Map of Site


Figure 2.

## PROCEDURE

### Tidal Regime

In the study area, tides are semi-diurnal, with 2 high and low waters occurring during each lunar day (approximately 24 hours and 50 minutes). The resulting astronomic tide range varies constantly in response to relative positions of the earth, moon, and sun, with the moon having the primary tide producing effect. Although long-term measurements are not available at the site, an approximation can be developed from correlation to historical tide data at the Boston, Massachusetts and Portland, Maine, National Ocean Survey gauges located 40 miles south and 60 miles north of the site, respectively. A summary of estimated tidal datums at the subject site is shown in Table 1 (taken from Little River Marsh Study, North Hampton and Hampton, New Hampshire; April 1999 US Army Corps of Engineers; New England District, Concord, Massachusetts).

**Tide Levels**

| | **Tide Level (ft, NGVD)** |
|---|---|
| Maximum Predicted Astronomical High Water | 6.7 |
| Mean High Water Spring | 5.2 |
| Mean High Water | 4.6 |
| Mean Tide Level | 0.45 |
| National Geodetic Vertical Datum (NGVD) | 0.0 |
| Mean Low Water | -3.7 |
| Mean Lower Low Water | -4.1 |
| Mean Low Water Spring | -4.4 |
| Minimum Predicted Astronomical Low Water | -5.9 |

Table 1.

The shape of the tidewater surface elevation is sinusoidal over time. Figure 3 is what the tide elevation would resemble when there is a mean high water spring elevation (5.2 ft NGVD) and mean lower spring elevation (-4.1 ft NGVD). The peaks were graphed in a 24-hour time period instead of the more precise measurement of a lunar day (24 hours and 50 minutes). This was done to make calculations easier.

Tide Levels

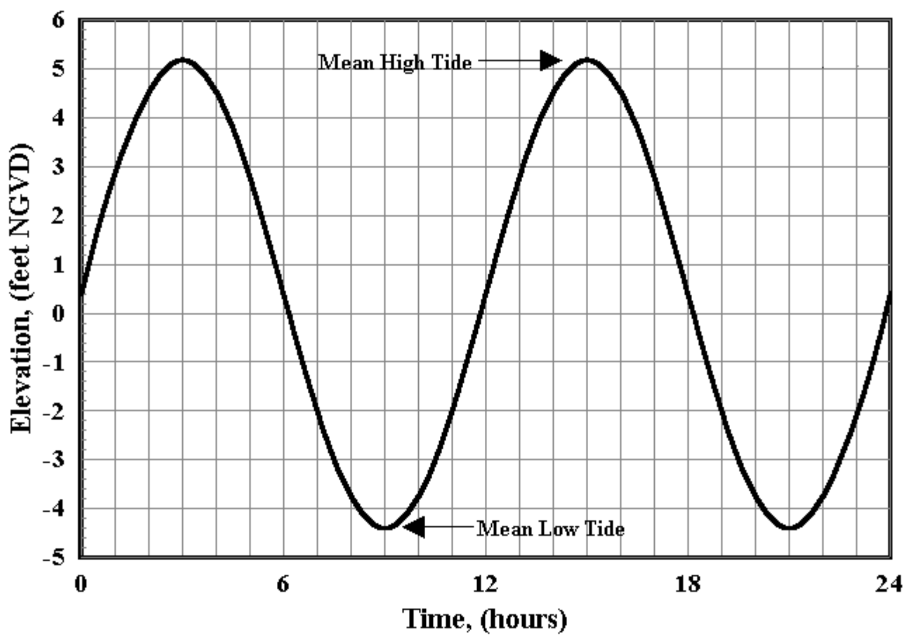



Figure 3.

## Surveying

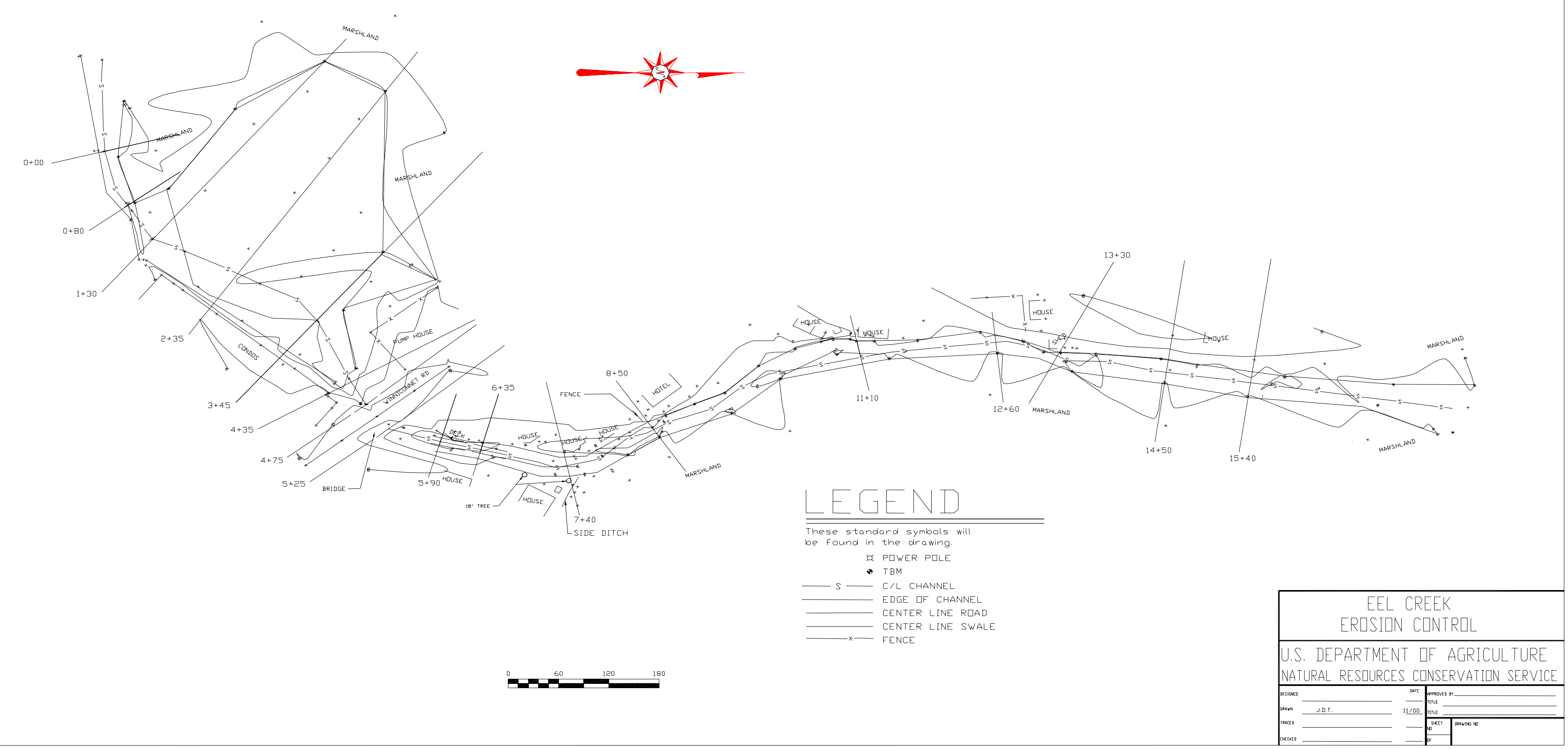
Eel Creek was surveyed on both the ocean and landside of the bridge. Figures 4 and 5 present the area that was surveyed. Cross sections of the channel and channel bottom elevations were recorded at selected locations. A high water mark with an elevation 4.9 ft (NGVD) was observed and recorded at the bridge. This elevation corresponds to the high water mark of 5.2 ft (NGVD) that the Army Corps predicted for Little River.

A more detailed survey of the creek and adjoining floodplain area will be needed to complete a detailed design of the proposed restoration project.

## Analysis

The water surface level (WSL) and velocities on the ocean side of the bridge were not computed. The ocean is controlling these conditions. However, the property on the landside of the bridge (referred to as Eel Creek in this paper) needed to be modeled. Modeling would determine the WSL and velocities at critical locations in the channel.

Figure 4. Survey Map



## Aerial Photo with Approximate Locations of Cross Sections

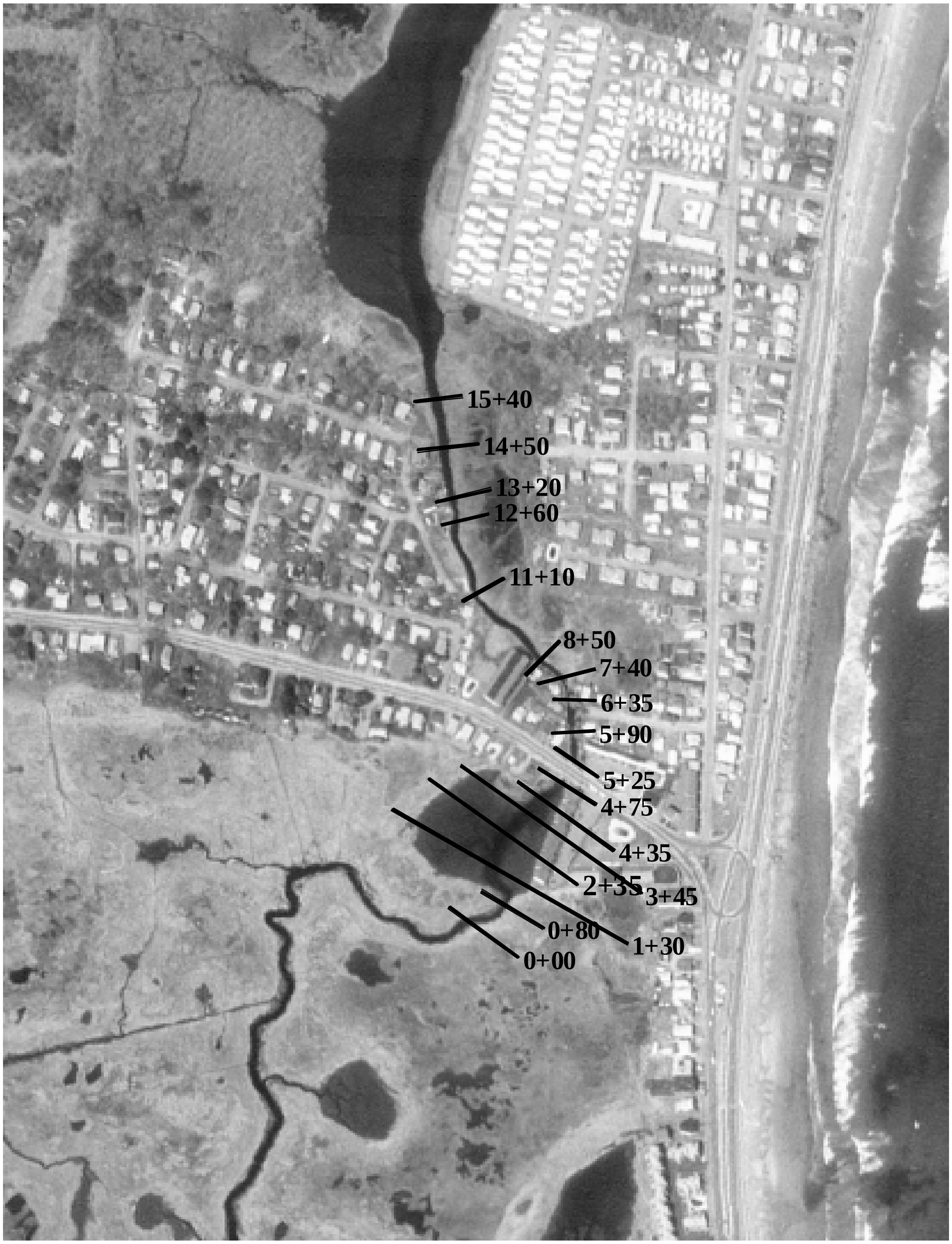



Figure 5.

## Modeling

Eel Creek was modeled using two different computer programs. The NRCS TR-20 computer program is a hydrology modeling program. The program computes direct runoff and peak discharges for storm events and routes the storm runoff through structures and streams. It was used to compute the peak discharges that flow into and out of Meadow Pond. The other program used was the USACOE HEC-RAS model, which is a hydraulic modeling program. It was used to calculate water surface levels (WSL) and velocities in Eel Creek.

The site was modeled to determine if the bridge is causing a restriction for both incoming and outgoing tides. First, Eel Creek was modeled without the bridge and then with the bridge. If the WSL and velocities remain the same, then the bridge would not cause a restriction.

Water velocities were calculated in Eel Creek to determine if the velocities are high enough to cause stream bank erosion. The velocities will be needed to design proper stream bank protections. Velocities and WSL were examined for four different scenarios.

The first scenario was to determine the greatest velocity when tidal water flows into Meadow Pond. Because tidal water is ponded in Meadow Pond rather than flowing over the marsh, modeling was difficult. The water levels in Meadow Pond and the tidal marsh continually change over time (tide cycle) and at different rates, which caused difficulty in modeling. The greatest velocity will occur when the elevation difference between the WSL of the incoming tide and the WSL in Meadow Pond is the largest. The greatest velocity probably occurs at mean high tide when the ocean would have an elevation at 5.2 ft NGVD and the WSL of the marsh at Meadow Pond would be 4.5 ft NGVD. It was determined that the WSL reaches an elevation of 4.5 ft NGVD on the marsh from the survey data. Cordgrass is growing at this elevation which typically grows in areas that are periodically inundated with tidal waters. A tide cycle survey would be needed to confirm this determination.

The second scenario was to calculate the maximum velocity and WSL when tidal water discharges from Meadow Pond into the ocean during low tide. The greatest velocity will occur when the difference in WSL between Meadow Pond and the ocean end of Eel Creek is largest. It was determined that the WSL in Meadow Pond is approximately 4.5 ft NGVD at high tide. When examining the sinusoidal curve of tide elevation (Figure 3), the elevation of the tide decreased in elevation approximately one foot per hour. Therefore, it was decided to use a tail water height of 3.5 ft for modeling low tide at the outlet of Eel Creek. Again, a tide cycle survey would be needed to document this determination.

A third scenario was to compute the velocity in Eel Creek when the peak discharge from a 1% chance (100-year) storm event occurs and Meadow Pond is at a mean high tide of 5.2 ft NGVD. Velocities were computed from discharge data at the bridge using HEC-RAS. A tail water depth of 5.2 ft was used as the boundary condition at the downstream end of the bridge. The computed elevation-discharge data was then used in TR-20 to determine the peak discharges from a storm event occurring at high tide in Meadow Pond. TR-20 was used to route storm flows through the bridge and compute the highest water surface level (WSL) and peak discharge. The maximum velocities at critical points along the creek were then computed.

The last scenario was to model the discharge from storm events when the ocean and Meadow Pond are at low tide. The storm events modeled were the 50% chance (2-year), 10% chance (10-year), 4% chance (25-year), 2% chance (50-year) and 1% chance (100-year) storm events. TR-20 was again used to route storm flows through Meadow Pond with the pond elevation at low tide. Another set of discharge data was created for low tide condition using HEC-RAS. Critical depth was used as the boundary condition at the downstream end of the bridge. Critical depth is the depth at which critical flow occurs. Critical flow is a term used in open channel flow analysis to define a dividing point between subcritical (tranquil) and supercritical (rapid) flow.

## STUDY RESULTS

### Eel Creek Results

The bridge had no effect on inflowing and outflowing water into and out of Meadow Pond. The elevations between bridge and no bridge simulations did differ by 0.1 ft. However, the 0.1 differences were sporadic. It was not constant through all cross-sections. This discrepancy may be attributed to differences in rounding. The average velocities did not change between bridge and non-bridge simulations.

### Tide Flow Results

The incoming tide is flowing from cross section 0+00 to 15+40. Table 2 and Figure 6 present the WSL and velocities of tidewaters for the incoming and outgoing tides into and out of Meadow Pond.

The incoming tide from the bridge to Meadow Pond has average velocities that range from 1.44 to 2.86 feet per second (fps). Cross sections 13+30 and 5+90 had the greatest velocity 2.86 fps and 2.67 fps respectively. These sections have smaller cross sectional areas as compared to the other cross sections, which causes the velocities to be higher.

The outgoing tide is flowing from cross section 15+40 to 0+00. When the outgoing tide flows from cross section 5+90 to 2+35, the channel bottom becomes much steeper. Water flowing down the steeper slope increases in velocity (approaching critical). Because of the steeper slope, the greatest velocity occurs at cross sections 5+90 with a velocity of 5.02 fps.

## Results of Incoming and Outgoing Tides

| Station (ft) | ($q_p = 160$ cfs) Incoming Tide WSL (ft, NGVD) | ($q_p = 160$ cfs) Incoming Tide Velocity (fps) | ($q_p = 100$ cfs) Outgoing Tide WSL (ft, NGVD) | ($q_p = 100$ cfs) Outgoing Tide Velocity (fps) |
|---|---|---|---|---|
| **Ocean** | | | | |
| 0+00 | 5.30 | 2.03 | 3.50 | |
| 0+80 | 5.30 | 1.27 | 3.52 | |
| 1+30 | 5.32 | 0.37 | 3.53 | 0.68 |
| 2+35 | 5.31 | 0.54 | 3.54 | 0.61 |
| 3+45 | 5.31 | 0.56 | 3.55 | 0.76 |
| 4+35 | 5.34 | 0.47 | 3.56 | 0.55 |
| 4+75 | 5.27 | 1.61 | 3.52 | 2.20 |
| **Center Line of Bridge** | | | | |
| 5+25 | 5.23 | 1.63 | 3.58 | 2.13 |
| 5+90 | 5.11 | 2.67 | 3.64 | 5.02 |
| 6+35 | 5.11 | 2.02 | 4.11 | 1.86 |
| 7+40 | 5.06 | 1.89 | 4.19 | 1.60 |
| 8+50 | 5.01 | 1.87 | 4.24 | 1.51 |
| 11+00 | 4.84 | 2.22 | 4.35 | 1.67 |
| 12+60 | 4.70 | 2.29 | 4.44 | 1.58 |
| 13+30 | 4.56 | 2.86 | 4.47 | 1.81 |
| 14+50 | 4.52 | 1.70 | 4.55 | 1.05 |
| 15+40 | 4.50 | 1.44 | 4.57 | 0.88 |
| **Meadow Pond** | | | | |

Table 2.

**Water Surface Elevation of Tide**
**(When Velocities are the Greatest)**

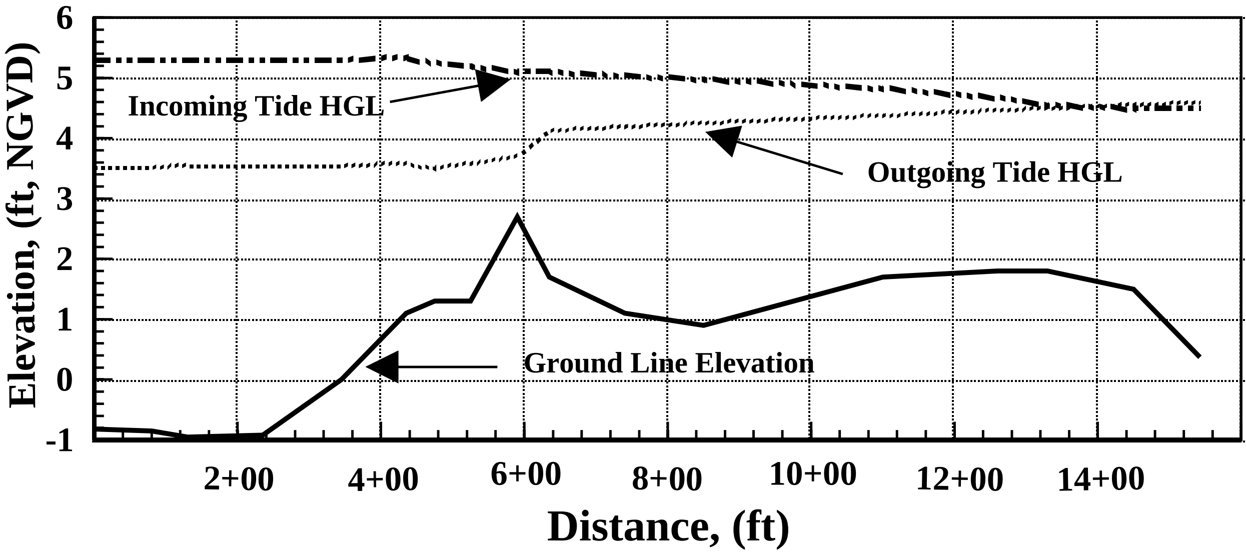



Figure 6.

**Results of Storm Events**

The velocities presented in Table 3 are the peak discharges from storm events in Meadow Pond at low tide. The water is flowing from Meadow Pond to the ocean. A boundary condition, critical depth, was used so that the model could calculate velocities. The model calibrates itself through several cross sections before it calculates realistic values. Therefore, the velocities at cross sections 0+00 and 0+80 were intentionally left blank because the computed velocities were unrealistic.

The greatest velocities for all storms analyzed occurred from cross section 5+90 to 4+75. The water increases in velocity because the channel bottom is steeper. When the peak flows reach cross section 3+45 the channel bottom has flatten and the depth of the water increases, a hydraulic jump probably occurs, because the velocity has decreased. When the water velocity decreases in this manner, it dissipates energy and the water surface levels (WSL) fluctuate. Since the WSL is not stable, it could be one of the reasons why erosion is taking place near the decks of the condominiums. However, it is not the cause of the suna tubes sinking.

## Present Condition Storm Event Peak Discharges Occurring at Low Tide

| Station (ft) | Storm Event Velocities (Peak Discharge) | | | | | | |
|---|---|---|---|---|---|---|---|
| | ($q_p$=123 cfs) 50% Chance (fps) | ($q_p$=200 cfs) 20% Chance (fps) | ($q_p$=240 cfs) 10% Chance (fps) | ($q_p$=405 cfs) 4% Chance (fps) | ($q_p$=476 cfs) 2% Chance (fps) | ($q_p$=619 cfs) 1% Chance (fps) | ($q_p$=100 cfs) Outgoing Tide Velocity (fps) |
| **Ocean** | | | | | | | |
| 0+00 | | | | | | | |
| 0+80 | | | | | | | |
| 1+30 | 1.42 | 1.82 | 2.00 | 2.57 | 2.77 | 3.12 | 0.68 |
| 2+35 | 1.18 | 1.56 | 1.73 | 2.29 | 2.49 | 2.84 | 0.61 |
| 3+45 | 1.69 | 2.07 | 2.24 | 2.78 | 2.97 | 3.30 | 0.76 |
| 4+35 | 2.17 | 1.81 | 1.77 | 1.79 | 1.83 | 1.89 | 0.55 |
| 4+75 | 5.49 | 6.43 | 6.85 | 8.16 | 8.61 | 9.39 | 2.20 |
| **Center Line of Bridge** | | | | | | | |
| 5+25 | 3.27 | 4.08 | 4.42 | 5.47 | 5.78 | 6.26 | 2.13 |
| 5+90 | 5.37 | 6.25 | 6.64 | 7.84 | 7.89 | 7.68 | 5.02 |
| 6+35 | 2.12 | 2.83 | 3.15 | 4.16 | 4.53 | 5.10 | 1.86 |
| 7+40 | 1.83 | 2.47 | 2.75 | 3.70 | 4.05 | 4.63 | 1.60 |
| 8+50 | 1.73 | 2.33 | 2.60 | 3.49 | 3.82 | 4.57 | 1.51 |
| 11+00 | 1.88 | 2.46 | 2.72 | 3.55 | 3.84 | 4.35 | 1.67 |
| 12+60 | 1.77 | 2.30 | 2.53 | 3.29 | 3.56 | 4.02 | 1.58 |
| 13+30 | 2.08 | 2.70 | 2.97 | 3.85 | 4.17 | 4.70 | 1.81 |
| 14+50 | 1.18 | 1.55 | 1.71 | 2.24 | 2.43 | 2.76 | 1.05 |
| 15+40 | 1.0 | 1.35 | 1.50 | 2.02 | 2.21 | 2.59 | 0.88 |
| **Meadow Pond** | | | | | | | |
| ($q_p$ = 727 cfs) Maximum Velocity of 1% Chance Storm Event Occurring at High Tide => | | | | | | 6.79 | |

Table 3.

Table 3 presents the velocities at cross sections for various storm events. The greatest velocities that occur upstream of the bridge are 5.37 fps for the 50% chance storm, 6.25 fps for the 20% chance storm, 6.64 fps for the 10% chance storm, 7.84 fps for the 4% chance storm, 7.89 fps for the 2% chance storm, and 7.68 fps for the 1% chance storm.

The cross sections used in the HEC-RAS model were divided into segments to more accurately show the water surface level (WSL) and velocities in the channel when water levels exceeded bank full elevations. The velocities in Table 3 from cross sections 6+35 to 15+40 (for the 10%, 4%, 2%, and 1% chance storms) are channel velocities. These channel velocities can be used for designing stream bank stabilization measures.

**Sensitivity Analysis**

A sensitivity analysis was performed on the HEC-RAS model. This is a technique to assess the relative change in a model response or output resulting from a change in inputs to the model. First, an analysis was performed to determine if the peak discharge increased when the elevation differences between the outlet and inlet is increased (for both incoming and outgoing tides). Any changes in flow were compared to the flows that were calculated for mean high and mean low tides (for the incoming and outgoing tides).

A second analysis was performed by varying Manning's "n" value. This coefficient is used for calculating the capacity of a channel to convey water. The coefficient was changed incrementally to determine how sensitive the peak flow (calculated for the mean high and mean low) for the incoming and outgoing tide was to changes in this coefficient.

**Results of Sensitivity Analysis**

The peak discharge flowing into Meadow Pond is not significantly affected by increasing the elevation difference between the ocean and the WSL in Meadow Pond. The peak discharge increased from 160 cfs to 180 cfs when the WSL elevation was lowered from 4.5 to 3.5 ft NGVD at Meadow Pond. This is a 1% increase of flow for one foot increase in elevation difference.

The peak discharge for outflow was not affected by an increase in elevation difference. The flow stayed constant at 100 cfs when the water elevation was lowered from 3.5 to 2.5 ft NGVD down stream of the bridge. Therefore, the channel is controlling the inflow and outflow discharges.

The model is sensitive to a change in Manning's "n" value. When the value was changed from 0.035 to 0.030 (decreasing the resistance to flow) for the incoming tide, the peak discharge increased from 160 to 190 cfs.

When the value was increased to 0.040 (increasing the resistance to flow) the peak discharge decreased from 160 to 140 cfs. When Manning's "n" was changed for the outgoing tide from 0.035 to 0.030 the peak discharge increased from 160 to 190 cfs. Conversely when Manning's "n" was increased from 0.035 to 0.040 the peak discharge decreased from 160 to 90 cfs. Realistically Manning's "n" would not be smaller than 0.035; therefore, the peak discharges would be 160 cfs or less for the incoming tide and 100 cfs or less for the outgoing tide.

## CONCLUSION

The bridge is not restricting daily inflow and outflow of tidewater. The maximum incoming and outgoing flows are 160 cfs and 100 cfs respectively. The mean high tide of the ocean is 5.2 ft NGVD. The corresponding outgoing high tide is approximately 4.5 ft NGVD in Meadow Pond. The maximum velocities for tidal flow occur at cross section 5+90. The velocities are 2.67 and 5.02 fps respectively for the incoming and outgoing tides.

The discharges from the storm events occurring at low tide have the greatest velocities (Table 3). The water abruptly changes velocity at cross section 5+90 because the channel bottom gets steeper and there is no tail water. Tail water adds resistance and therefore slows down the velocity.

The properties upstream of the bridge that have stream bank erosion problems need to be stabilized to withstand a minimum velocity of 5.02 fps at a depth of 3.6 ft. Table 3 presents the velocities that occur during a storm event. These velocities are greater than the tide velocities; therefore, they should be considered when making decisions on stream bank stabilization.

## POSSIBLE ALTERNATIVES

The following is a list of possible stream bank protection solutions. It is imperative to note that any existing cordgrass and any cordgrass that establishes itself in the future should not be disturbed. Cordgrass is nature's way of stabilizing salt marsh channel banks. Once established, it protects stream banks from the high water velocities that naturally occur in tidal salt marshes.

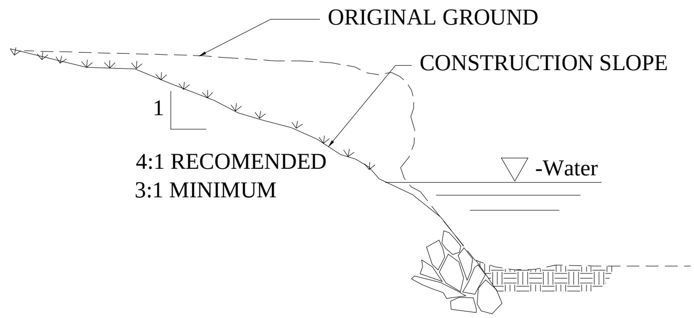
The estimated cost for all of the listed alternatives (Table 6), the address of the property owners (Table 7), and the total disturbed area (Table 8) are located at the end of this report.

### Alternative 1 – Do Nothing

One possibility is to do nothing and over time the channel will erode to a stable condition. However, this does not solve the current problem, and will have a major impact to houses along Eel Creek.

**Alternative 2 – Riprap Toe**

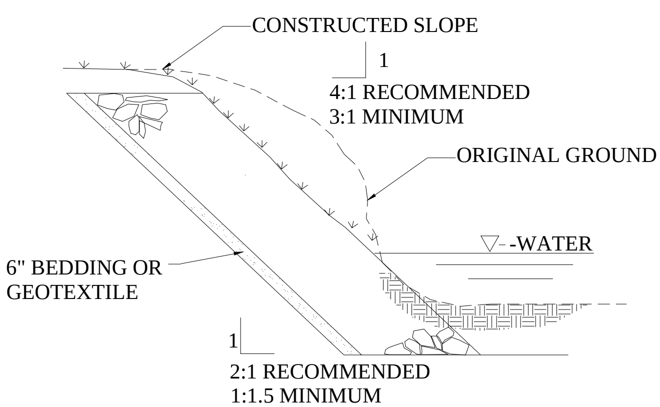
This alternative would stabilize the stream toe with riprap (Figure 7). Any bare soil above the rock could be shaped and protected with erosion control matting. The matting would protect the soil until the salt marsh cordgrass could establish itself. It may take several years for the grass to naturally establish.




**Stabilizing Stream Toe with Rock Riprap**
Figure 7.

**Alternative 3 – Riprap Banks**

This alternative is to place rock riprap on the banks from the toe to the elevation near high tide (Figure 8). The approximate height of the riprap would be 4.5 ft. This method would make the banks more stable than just ripraping the toe. After the rocks trap enough sediment, the cordgrass would grow through the voids. Thus the grass would camouflage the rock. The disadvantage of this approach is that it would be more expensive than just ripraping the toe.




**Stabilizing Stream Toe and Bank with Rock Riprap**
Figure 8.

**Alternative 4 – Sheet Piling**

This alternative requires lining the channel with vinyl sheet pile (Figure 9). Vinyl sheet pile has the advantage over traditional sheet pile that it can be installed without the use of large heavy equipment. However, it may be the most expensive alternative. It does have the advantage that it can be installed wherever space limitations make stabilizing stream banks difficult. Plus it maintains the largest cross sectional area for flowage.

**Stabilizing Bank With Sheet Pile**

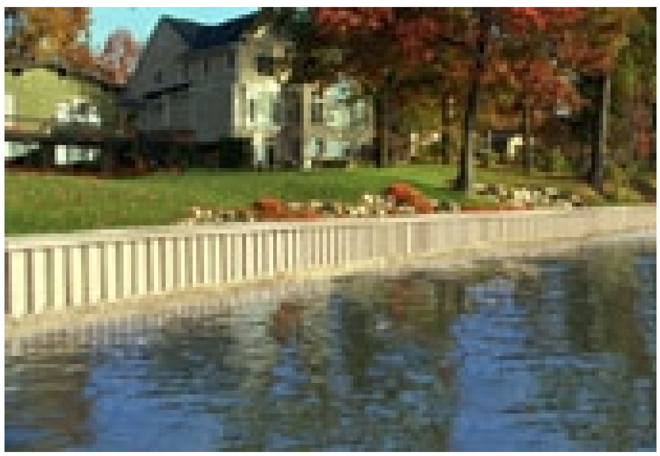


Figure 9.

Instead of making a retaining wall out of vinyl sheet pile, it could be constructed out of wood posts. The posts would be installed side by side when driven into the ground. The price for a pressure treated wood post retaining wall is approximately $386 per foot. Using wood posts instead of vinyl is approximately three times more expensive.

**Alternative 5 – Dredging and Stabilizing the Stream Banks**

Another alternative is to dredge the channel bottom from cross section 4+35 to 6+35. The channel bottom would be dredged to have a uniform inclined grade (Figure 10) and a uniform width of 24 ft (the width of the bridge). In addition to dredging, some bank stabilization method will be needed at critical locations.

This method would eliminate the high velocities and turbulence caused by the outgoing tide and storm discharges. The peak velocities at cross section 5+90 for the listed storm events would be reduced (approximately) in half and the peak discharge of tide water (and velocity) into Meadow Pond would increase slightly.

However, the increase is insignificant and the added flow would help improve Meadow Pond as a salt marsh environment.

Because of the dredging, the permanent pool level of Meadow Pond will be reduced approximately 1.5 ft. Therefore, parts of Meadow Pond will be mud flats during low tide. It will probably take several years for cordgrass to establish on the mud flats.

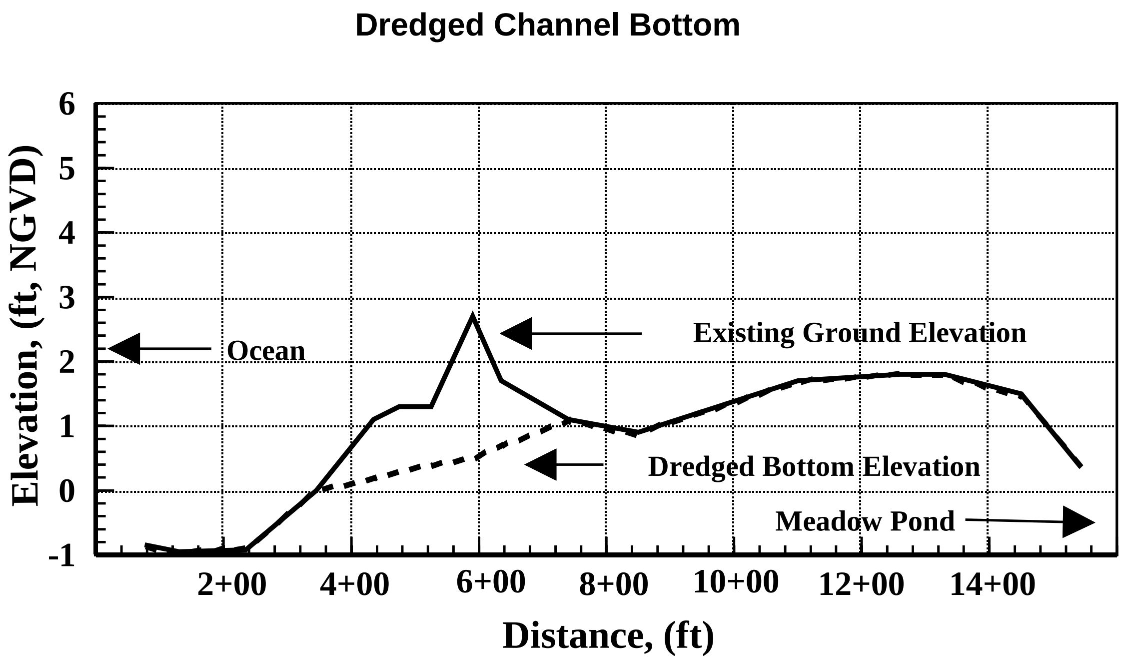



Figure 10.

**RECOMMENDED ALTERNATIVE**

It is recommended that Eel Creek be deepened from cross section 3+45 to 4+35 and widened and deepened from cross section 4+35 to 7+40. This will give the creek a more uniform grade and width that is more representative of a natural tidal creek. This alternative would eliminate the high velocities caused by the outgoing tide (Table 4) and storm discharges (Table 5). The greatest velocity (at cross section 5+90) would be reduced from 5.02 to 1.74 fps for the outgoing tide.

It is also recommended that the banks be stabilized with either riprap or sheet piling in areas where continual erosion threatens developed property. Stabilizing the bank with sheet pile is more expensive than stabilizing with rock riprap. However, riprap requires more area than sheet pile to stabilize a bank. Therefore, where space is a constraint, sheet pile is recommended.

## Present Condition and Recommended Alternative Tide Velocities

| Station (ft) | (q = 160 cfs) Present Condition Incoming Tide (fps) | (q = 190 cfs) Recommended Alternative Tide (fps) | (q = 100) Present Condition Outgoing Tide (fps) | (q = 130 cfs) Recommended Alternative Outgoing Tide (fps) |
|---|---|---|---|---|
| **Ocean** | | | | |
| 0+00 | 2.03 | 2.41 | | |
| 0+80 | 1.27 | 1.51 | | |
| 1+30 | 0.37 | 0.49 | 0.68 | 0.99 |
| 2+35 | 0.54 | 0.72 | 0.61 | 0.87 |
| 3+45 | 0.56 | 0.74 | 0.76 | 1.08 |
| 4+35 | 0.47 | 1.28 | 0.55 | 0.48 |
| 4+75 | 1.63 | 1.13 | 2.20 | 1.42 |
| **Center Line of Bridge** | | | | |
| 5+25 | 1.65 | 1.63 | 2.13 | 1.70 |
| 5+90 | 2.67 | 1.67 | 5.02 | 1.74 |
| 6+35 | 2.02 | 1.74 | 1.86 | 1.86 |
| 7+40 | 1.89 | 2.18 | 1.60 | 2.58 |
| 8+50 | 1.87 | 2.07 | 1.51 | 2.27 |
| 11+00 | 2.22 | 2.42 | 1.67 | 2.39 |
| 12+60 | 2.29 | 2.48 | 1.58 | 2.06 |
| 13+30 | 2.86 | 3.18 | 1.81 | 2.40 |
| 14+50 | 1.70 | 1.95 | 1.05 | 1.35 |
| 15+40 | 1.44 | 1.65 | 0.88 | 1.13 |
| **Meadow Pond** | | | | |

Table 4.

## Recommended Alternative Peak Discharges for Storm Events Occurring at Low Tide

| Station | Storm Event Velocities (Peak Discharge) | | | | | | ($q_p$ = 130 cfs) Outgoing Tide Velocity (fps) |
|---|---|---|---|---|---|---|---|
| | ($q_p$ = 123 cfs) 50% Chance (fps) | ($q_p$ = 200 cfs) 20% Chance (fps) | ($q_p$ = 240 cfs) 10% Chance (fps) | ($q_p$ = 405 cfs) 4% Chance (fps) | ($q_p$ = 476 cfs) 2% Chance (fps) | ($q_p$ = 619 cfs) 1% Chance (fps) | |
| **Ocean** | | | | | | | |
| 0+00 | | | | | | | |
| 0+80 | | | | | | | |
| 1+30 | 1.56 | 2.04 | 2.25 | 2.92 | 3.15 | 3.56 | 0.99 |
| 2+35 | 1.32 | 1.75 | 1.93 | 2.52 | 2.72 | 3.07 | 0.87 |
| 3+45 | 1.88 | 2.30 | 2.47 | 3.03 | 3.22 | 3.54 | 1.08 |
| 4+35 | 0.93 | 1.04 | 1.09 | 1.27 | 1.33 | 1.45 | 0.48 |
| 4+75 | 2.53 | 3.05 | 3.26 | 3.93 | 4.15 | 4.59 | 1.42 |
| **Center Line of Bridge** | | | | | | | |
| 5+25 | 2.82 | 3.47 | 3.75 | 4.69 | 5.00 | 5.52 | 1.70 |
| 5+90 | 2.78 | 3.42 | 3.69 | 4.62 | 4.94 | 5.45 | 1.74 |
| 6+35 | 2.96 | 3.59 | 3.86 | 4.78 | 5.09 | 5.55 | 1.86 |
| 7+40 | 4.36 | 4.85 | 5.07 | 5.86 | 6.13 | 6.58 | 2.58 |
| 8+50 | 2.73 | 3.38 | 3.65 | 4.46 | 4.70 | 5.05 | 2.27 |
| 11+00 | 2.61 | 3.20 | 3.45 | 4.26 | 4.52 | 4.86 | 2.39 |
| 12+60 | 2.17 | 2.65 | 2.85 | 3.49 | 3.71 | 4.14 | 2.06 |
| 13+30 | 2.50 | 3.07 | 3.31 | 4.03 | 4.23 | 4.63 | 2.40 |
| 14+50 | 1.38 | 1.73 | 1.88 | 2.39 | 2.57 | 2.91 | 1.35 |
| 15+40 | 1.13 | 1.48 | 1.63 | 2.13 | 2.31 | 2.65 | 1.13 |
| **Meadow Pond** | | | | | | | |

Table 5.

The area in front of the condominiums, located between cross sections 1+30 and 4+75, should be stabilized with sheet pile. The sheet pile should be installed a distance of three feet away from the decking of the condominiums. To keep the sheet piling from being undermined by wave action, rock rip should be placed in front of it. Ripraping the bank to an elevation of 4.5 ft is recommended on the opposite side of the stream from cross 3+90 to 4+75. This will protect the property on the other side.

The locations of the houses on the north side of Winnacunnet Road that border Eel Creek from cross section 5+25 to 7+40 also affect the type of stream protection that can be used. Sheet piling has the advantage of maintaining a 24 ft bottom width while minimizing losses of personal property to stream channel area. The stream bank (on the West Side) should be protected with sheet piling from cross section 7+40 to 8+00. The distance between the houses and the toe (cross sections 7+40 to 8+00) make riprap impractical. The remaining reach of the stream bank at this location (on the West Side from cross section 8+00 to 9+20) can be stabilized with the use of rock riprap protecting the toe.

The properties from cross section 10+60 to 11+60 can be protected with rock riprap. However, the bank at certain locations must be reshaped into Eel Creek with the use of fill. For safety, this should be done where there is less than five feet of bank between the house and top of stream bank. The stream banks should be protected from the toe to the top of the bank with riprap.

The other building that is being threatened by stream bank erosion is the shed located downstream of cross section 13+30. It is recommend that this building be moved to another location. By moving the shed, it will no longer be endangered. If for some reason it can’t be moved, the toe of the stream bank should be stabilized with riprap for a length of 20 ft both upstream and downstream from the shed.

## Estimated Bank Stabilization Costs

| STARTING CROSS SECTION | ENDING CROSS SECTION | RIP-RAP TOE | RIP-RAP TOP OF BANK | SHEET PILING | DREDGING | RECOMMENDED ALTERNATIVE | EAST SIDE RECOMMENDED ALTERNATIVE | WEST SIDE RECOMMENDED ALTERNATIVE |
|---|---|---|---|---|---|---|---|---|
| 0+00 | 0+80 | N/A | N/A | N/A | N/A | N/A | N/A | N/A |
| 0+80 | 1+30 | N/A | N/A | N/A | N/A | N/A | N/A | N/A |
| 1+30 | 2+35 | $4,400 | $5,400 | $15,900 | N/A | $15,900 | Sheet Pile | Nothing |
| 2+35 | 3+45 | $4,400 | $5,700 | $16,600 | N/A | $16,600 | Sheet Pile | Nothing |
| 3+45 | 3+90 | $3,200 | $3,500 | $13,300 | $200 | $6,900 | Sheet Pile<br>Dredging | Nothing |
| 3+90 | 4+35 | $4,300 | $4,700 | $13,500 | $200 | $9,300 | Sheet Pile<br>Dredging | RR Bank |
| 4+35 | 4+75 | $4,200 | $5,000 | $11,700 | $600 | $5,600 | RR Bank<br>Dredging | RR Bank<br>Dredging |
| 4+75 | 5+25 | Loca-tion of Bridge | Loca-tion of Bridge | Loca-tion of Bridge | $500 | $500 | Dredging | Dredging |
| 5+25 | 5+90 | X | X | $19,000 | $1,200 | $20,200 | Sheet Pile<br>Dredging | Sheet Pile<br>Dredging |
| 5+90 | 6+35 | X | X | $12,900 | $700 | $13,600 | Sheet Pile<br>Dredging | Sheet Pile<br>Dredging |
| 6+35 | 7+40 | X | X | $29,700 | $700 | $30,600 | Sheet Pile<br>Dredging | Sheet Pile<br>Dredging |
| 7+40 | 8+50 | X | X | $15,600 | N/A | $15,600 | Nothing | Sheet Pile |
| 8+50 | 9+20 | $2,300 | $3,000 | $9,900 | N/A | $2,800 | Nothing | RR Toe |
| 9+20 | 10+60 | $4,000 | $5,900 | $19,800 | N/A | | Nothing | Nothing |
| 10+60 | 11+10 | X | $2,100 | $7,100 | N/A | $2,000 | Nothing | Fill<br>RR Bank |
| 11+10 | 11+60 | X | $2,100 | $7,100 | N/A | $2,000 | Nothing | Fill<br>RR Bank |
| 11+60 | 12+60 | $2,900 | $4,100 | $14,100 | N/A | $4,400 | Nothing | Nothing |
| 12+60 | 13+30 | $2,400 | $2,900 | $9,900 | N/A | | Nothing | Nothing |
| 13+30 | 14+50 | $3,800 | $4,900 | $17,000 | N/A | | Nothing | Nothing |
| 14+50 | 15+40 | $2,800 | $3,800 | $12,700 | N/A | | Nothing | Nothing |
| | | | | | **Mobilization Cost =>** | **$14,600** | | |
| | | | | | **Contingencies Cost =>** | **$29,200** | | |
| | | | | | **Engineering Services =>** | **$14,600** | | |
| | | | | | **Administration Cost =>** | **$21,900** | | |
| | | | | | **TOTAL COST =>** | **$226,300** | | |

Note: X denotes area where this bank stabilization method is not recommended.

Table 6.

## Properties Bordering Eel Creek

| Starting Cross Section | Ending Cross Section | Address of Owner West Side of Eel Creek | Address of Owner East Side of Eel Creek |
|---|---|---|---|
| 3+45 | 4+35 | Town of Hampton<br>NYNEX | 571 Winnacunnet Road |
| 4+35 | 4+75 | Town of Hampton<br>NYNEX | 571 Winnacunnet Road |
| 4+75 | 5+25 | Winnacunnet Road | Winnacunnet Road |
| 5+25 | 5+90 | 566 Winnacunnet Road | 580 Winnacunnet Road |
| 5+90 | 6+35 | 566 Winnacunnet Road | 580 Winnacunnet Road<br>15 Red Coat Lane |
| 6+35 | 7+40 | 566 Winnacunnet Road<br>554 Winnacunnet Road | 15 Red Coat Lane<br>14 Red Coat Lane |
| 7+40 | 8+50 | 554 Winnacunnet Road<br>550 Winnacunnet Road | Lot # 99<br>Marsh |
| 8+50 | 11+10 | 550 Winnacunnet Road<br>1 Thorwald Street<br>18 Sapphire Avenue | Marsh |
| 11+10 | 12+60 | 18 Sapphire Avenue<br>Lot # 74<br>Lot # 66 | Marsh |
| 12+60 | 13+30 | Lot # 66<br>8 Sapphire Avenue<br>6 Sapphire Avenue | Marsh |
| 13+30 | 14+50 | 6 Sapphire Avenue<br>4 Sapphire Avenue<br>2 Sapphire Avenue | Marsh |
| 14+50 | 15+40 | 2 Sapphire Avenue<br>5 Walnut Street | Marsh |

The addresses were taken from the Property Assessment Map for the Town of Hampton. Lot # 's had no address listed on the map.

Table 7.

## DISTURBED AREAS

| Start Cross-Section | End Cross-Section | Disturbed Area[1] ($ft^2$) | Dredged Area[2] ($ft^2$) | Fill Area[3] ($ft^2$) |
|---|---|---|---|---|
| 0+00 | 0+80 | 0 | 0 | 0 |
| 0+80 | 1+30 | 0 | 0 | 0 |
| 1+30 | 2+35 | 420 | 0 | 420 |
| 2+35 | 3+45 | 440 | 0 | 440 |
| 3+45 | 3+90 | 1,598 | 1,125 | 473 |
| 3+90 | 4+35 | 2,160 | 1,125 | 1,035 |
| 4+35 | 4+75 | 1,080 | 500 | 580 |
| 4+75 | 5+25 | 960 | 960 | 0 |
| 5+25 | 5+90 | 2,145 | 1,560 | 585 |
| 5+90 | 6+35 | 1,485 | 1,080 | 405 |
| 6+35 | 7+40 | 2,573 | 1,418 | 1,155 |
| 7+40 | 8+50 | 990 | 165 | 825 |
| 8+50 | 9+20 | 175 | 0 | 175 |
| 9+20 | 10+60 | 1,120 | 0 | 1,120 |
| 10+60 | 11+10 | 625 | 0 | 625 |
| 11+10 | 11+60 | 440 | 0 | 440 |
| 11+60 | 12+60 | 0 | 0 | 0 |
| 12+60 | 13+30 | 0 | 0 | 0 |
| 13+30 | 14+50 | 0 | 0 | 0 |
| 14+50 | 15+40 | 0 | 0 | 0 |

[1] Disturbed Area is the total area disturbed by dredging and filling.

[2] Dredged Area is the area of the channel that is going to be widened and/or deepened.

[3] Fill Area is the area that rock riprap or stone fill material is placed on the channel bank and/or in the channel

Table 8.

# APPENDIXES

**Appendix A -- Cost for Alternative 2 (Riprap Toe)**

**Appendix B -- Cost for Alternative 3  (Riprap Banks)**

**Appendix C -- Cost for Alternative 4  (Sheet Pile)**

**Appendix D --Price of Quantities**

**Appendix E -- Cross Sections**

## Appendix A -- Cost for Alternative 2 (Riprap Toe)

| Start X-Sec | End X-Sec | Excavated Cost | Riprap Costs | Fill Costs | Geo Costs | Total Costs |
|---|---|---|---|---|---|---|
| 0+00 | 0+80 | $0 | $0 | $0 | $0 | $0 |
| 0+80 | 1+30 | $0 | $0 | $0 | $0 | $0 |
| 1+30 | 2+35 | $1,350 | $2,730 | $0 | $232 | $4,400 |
| 2+35 | 3+45 | $1,400 | $2,730 | $0 | $241 | $4,400 |
| 3+45 | 3+90 | $300 | $2,730 | $0 | $97 | $3,200 |
| 3+90 | 4+35 | $0 | $4,095 | $0 | $193 | $4,300 |
| 4+35 | 4+75 | $900 | $3,120 | $0 | $174 | $4,200 |
| 4+75 | 5+25 | $0 | $0 | $0 | $0 | $0 |
| 5+25 | 5+90 | $1,300 | $1,755 | $780 | $145 | $4,000 |
| 5+90 | 6+35 | $900 | $1,170 | $585 | $97 | $2,800 |
| 6+35 | 7+40 | $1,600 | $2,145 | $780 | $232 | $4,800 |
| 7+40 | 8+50 | $1,150 | $1,950 | $390 | $241 | $3,800 |
| 8+50 | 9+20 | $700 | $1,365 | $0 | $154 | $2,300 |
| 9+20 | 10+60 | $1,100 | $2,535 | $0 | $309 | $4,000 |
| 10+60 | 11+10 | $300 | $975 | $0 | $116 | $1,400 |
| 11+10 | 11+60 | $300 | $975 | $0 | $87 | $1,400 |
| 11+60 | 12+60 | $900 | $1,755 | $0 | $241 | $2,900 |
| 12+60 | 13+30 | $800 | $1,365 | $0 | $154 | $2,400 |
| 13+30 | 14+50 | $1,350 | $2,145 | $0 | $261 | $3,800 |
| 14+50 | 15+40 | $1,000 | $1,560 | $0 | $193 | $2,800 |

## Appendix B -- Cost for Alternative 3 (Riprap Banks)

| Start X-Sec | End X-Sec | Excavated Cost | Riprap Costs | Fill Costs | Geo Costs | Total Costs |
|---|---|---|---|---|---|---|
| 0+00 | 0+80 | $0 | $0 | $0 | $0 | $0 |
| 0+80 | 1+30 | $0 | $0 | $0 | $0 | $0 |
| 1+30 | 2+35 | $1,250 | $3,900 | $0 | $232 | $5,400 |
| 2+35 | 3+45 | $1,300 | $4,095 | $0 | $241 | $5,700 |
| 3+45 | 3+90 | $450 | $2,925 | $0 | $97 | $3,500 |
| 3+90 | 4+35 | $350 | $3,900 | $195 | $193 | $4,700 |
| 4+35 | 4+75 | $1,050 | $3,510 | $195 | $174 | $5,000 |
| 4+75 | 5+25 | $0 | $0 | $0 | $0 | $0 |
| 5+25 | 5+90 | $1,150 | $1,950 | $1,560 | $145 | $4,900 |
| 5+90 | 6+35 | $750 | $1,560 | $585 | $97 | $3,000 |
| 6+35 | 7+40 | $1,400 | $3,315 | $780 | $232 | $5,800 |
| 7+40 | 8+50 | $1,100 | $3,120 | $390 | $241 | $4,900 |
| 8+50 | 9+20 | $650 | $1,950 | $195 | $154 | $3,000 |
| 9+20 | 10+60 | $650 | $3,900 | $975 | $309 | $5,900 |
| 10+60 | 11+10 | $0 | $1,365 | $585 | $116 | $2,100 |
| 11+10 | 11+60 | $0 | $1,365 | $585 | $116 | $2,100 |
| 11+60 | 12+60 | $550 | $2,730 | $585 | $222 | $4,100 |
| 12+60 | 13+30 | $750 | $1,950 | $0 | $154 | $2,900 |
| 13+30 | 14+50 | $1,250 | $3,315 | $0 | $270 | $4,900 |
| 14+50 | 15+40 | $1,000 | $2,535 | $0 | $193 | $3,800 |

## Appendix C -- Cost for Alternative 4 (Sheet Pile)

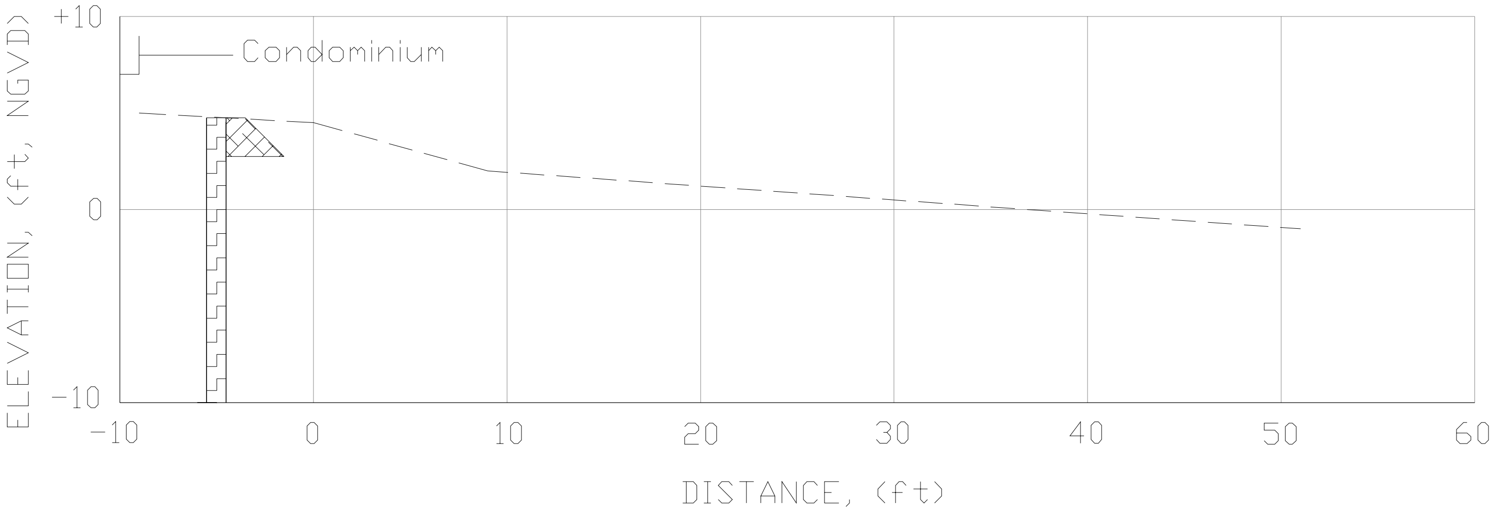
| Start X-Sec | End X-Sec | ill Price | Sheet Pile | Total Costs |
|---|---|---|---|---|
| 0+00 | 0+80 | $0 | $0 | $0 |
| 0+80 | 1+30 | $0 | $0 | $0 |
| 1+30 | 2+35 | $1,000 | $14,900 | $15,900 |
| 2+35 | 3+45 | $1,000 | $15,600 | $16,600 |
| 3+45 | 3+90 | $600 | $12,700 | $13,300 |
| 3+90 | 4+35 | $800 | $12,700 | $13,500 |
| 4+35 | 4+75 | $400 | $11,300 | $11,700 |
| 4+75 | 5+25 | $0 | $0 | $0 |
| 5+25 | 5+90 | $600 | $18,400 | $19,000 |
| 5+90 | 6+35 | $200 | $12,700 | $12,900 |
| 6+35 | 7+40 | $0 | $29,700 | $29,700 |
| 7+40 | 8+50 | $0 | $15,600 | $15,600 |
| 8+50 | 9+20 | $0 | $9,900 | $9,900 |
| 9+20 | 10+60 | $0 | $19,800 | $19,800 |
| 10+60 | 11+10 | $0 | $7,100 | $7,100 |
| 11+10 | 11+60 | $0 | $7,100 | $7,100 |
| 11+60 | 12+60 | $0 | $14,100 | $14,100 |
| 12+60 | 13+30 | $0 | $9,900 | $9,900 |
| 13+30 | 14+50 | $0 | $17,000 | $17,000 |
| 14+50 | 15+40 | $0 | $12,700 | $12,700 |

## Appendix D -- Price of Quantities

| Item | Cost | Description<br>(Source of Price) |
|---|---|---|
| Excavating | $7/yd^3$ | Cost of Excavating Channel (Little River Project) |
| Hauling | $40/15yd^3$ | Cost of Truck for 1 hour (Little River Project) |
| Geotextile | $1.93/yd^2$ | (RSMeans Heavy Construction Cost Data) |
| Sheet Piling | $141/ft | (RSMeans Heavy Construction Cost Data) |
| Wood Post Retaining Wall | $386/ft | (RSMeans Heavy Construction Cost Data) |
| Riprap | $39/yd^3$ | Cost of rock; hauling; machined placed<br>(RSMeans Heavy Construction Cost Data) |
| Fill | $39/yd^3$ | Cost of rock; hauling; machined placed<br>(RSMeans Heavy Construction Cost Data) |

# Appendix E --Cross Sections

# CROSS SECTION 1+30

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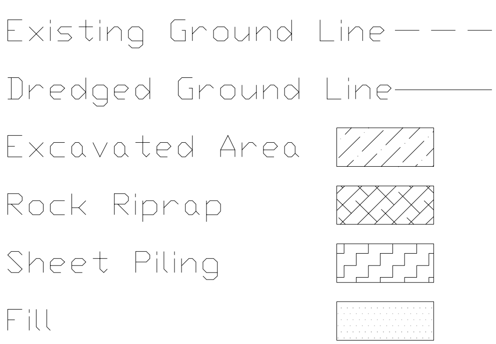


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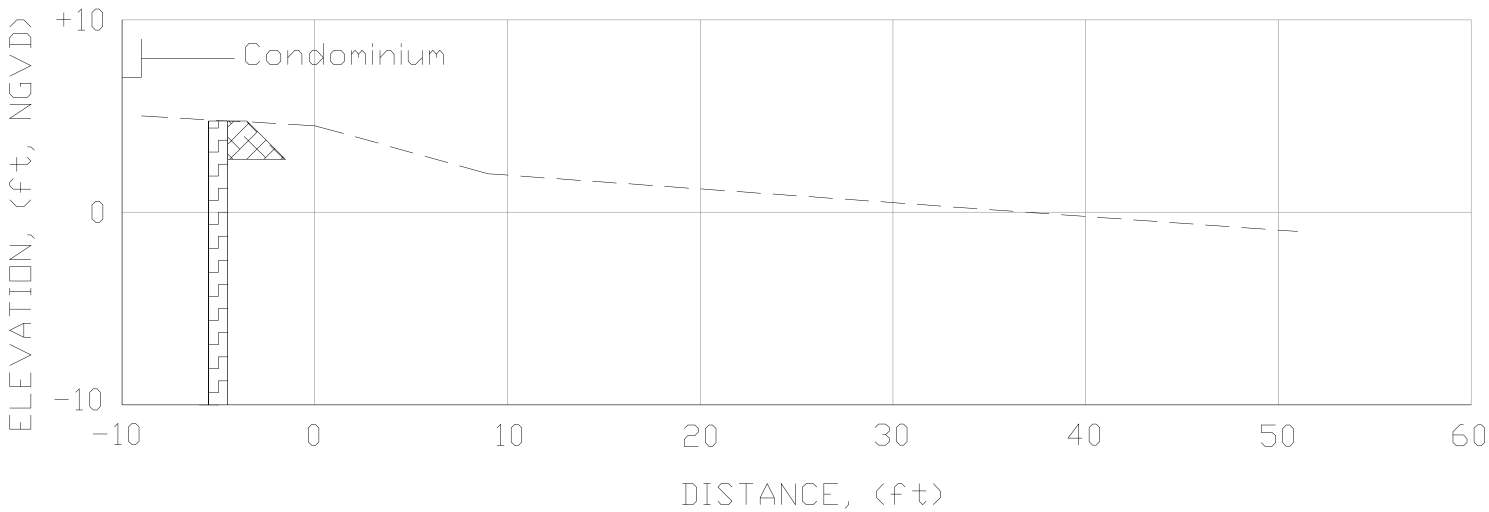
# CROSS SECTION 2+35





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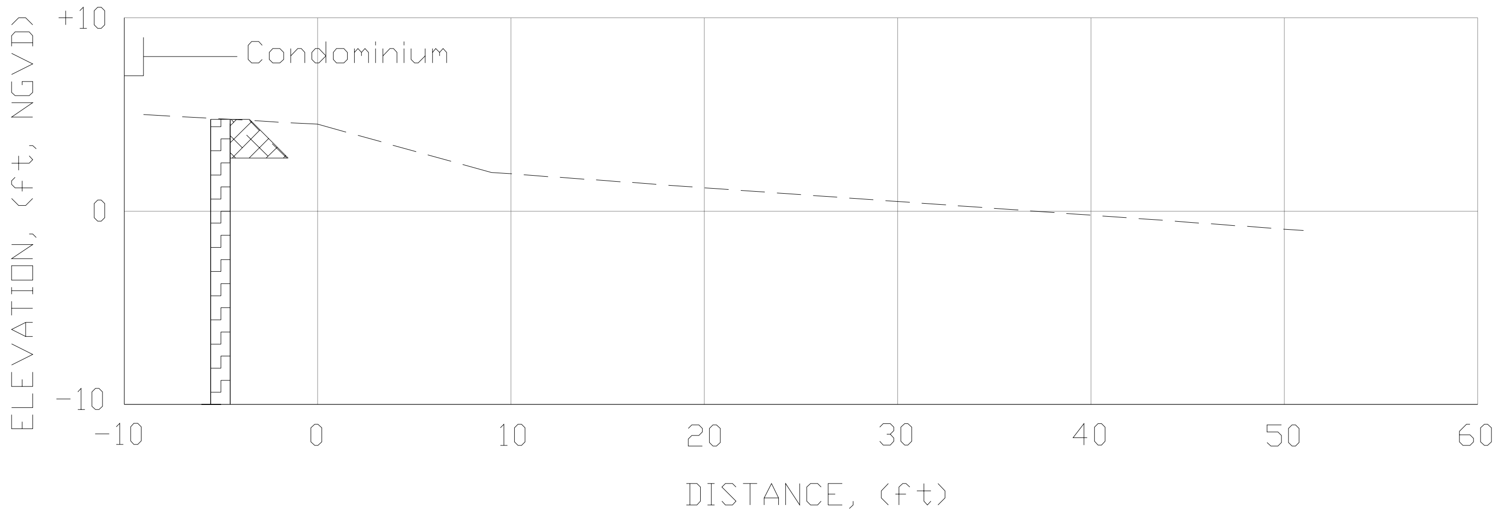
## CROSS SECTION 3+45

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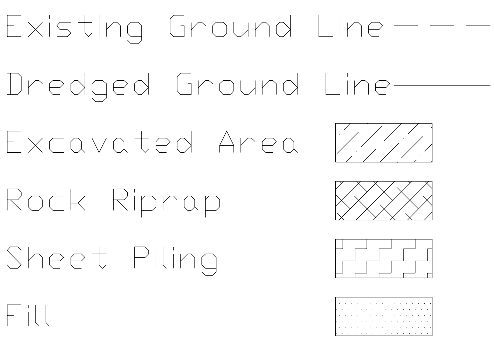


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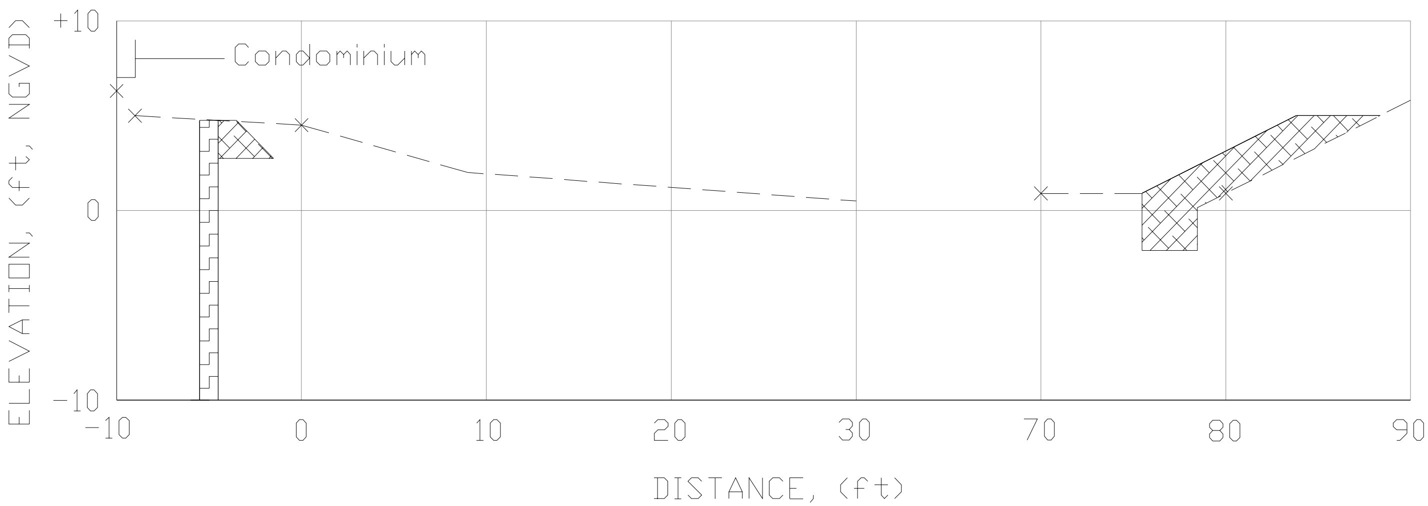
CROSS SECTION 3+90





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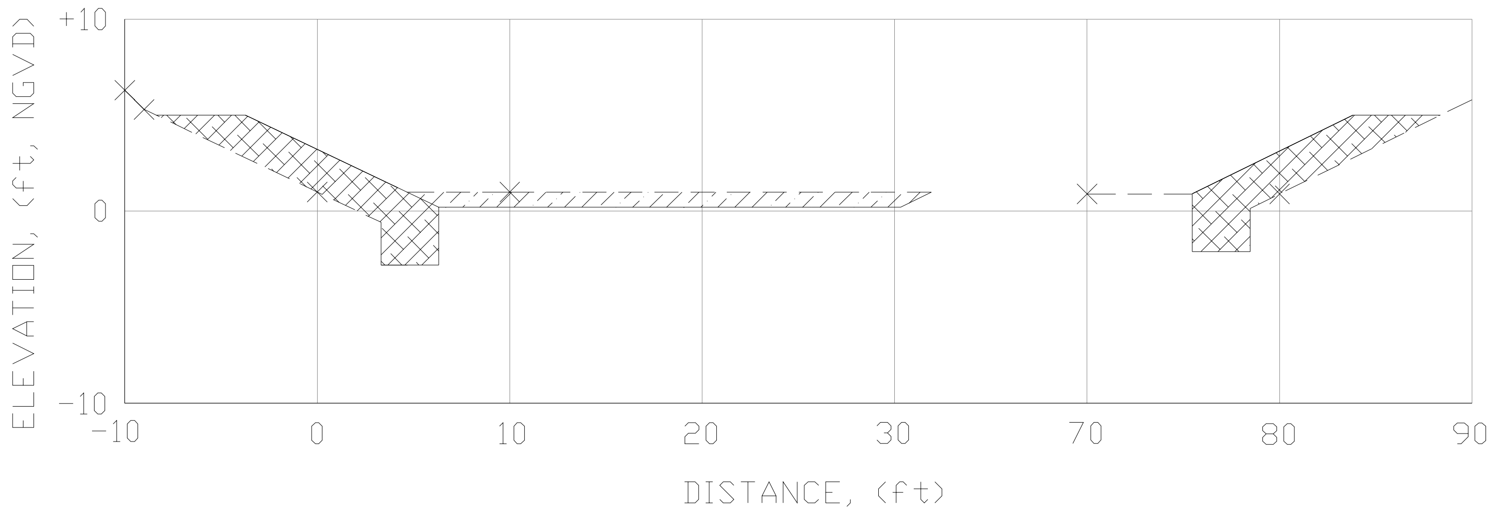


LEGEND

Existing Ground Line — — —

Dredged Ground Line ———

Excavated Area

Rock Riprap

Sheet Piling

Fill

PROPOSED ALTERNATIVE
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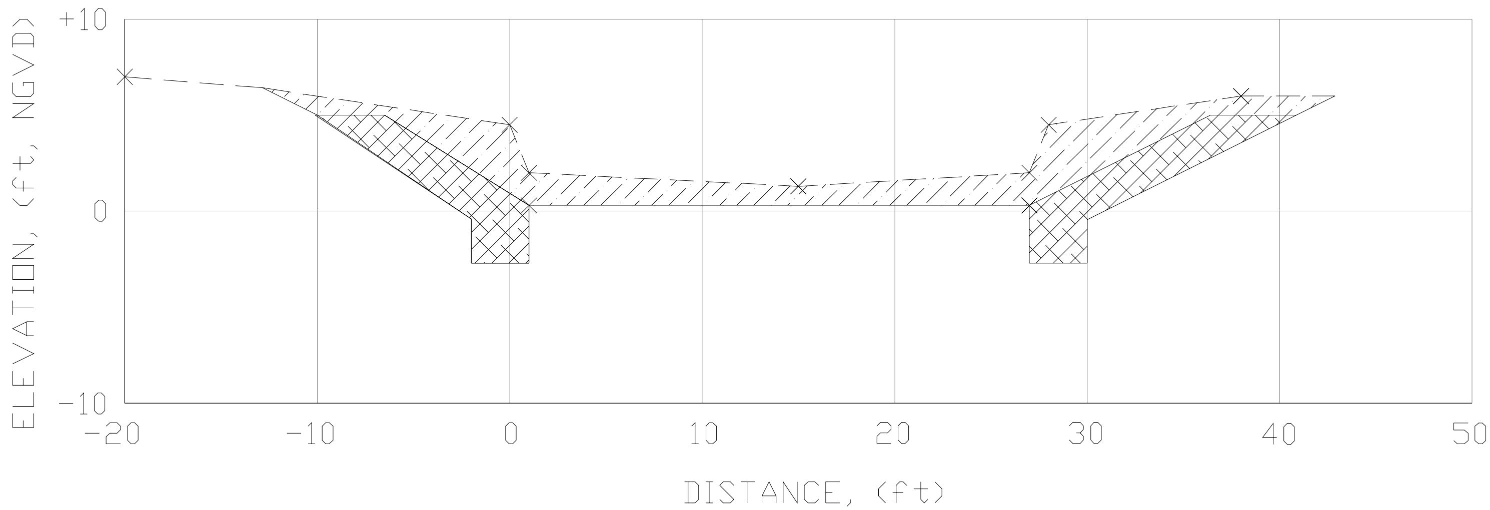
## CROSS SECTION 4+75

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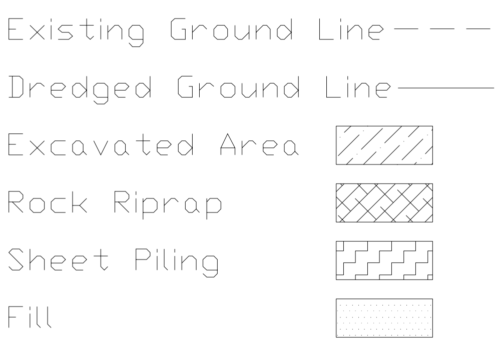


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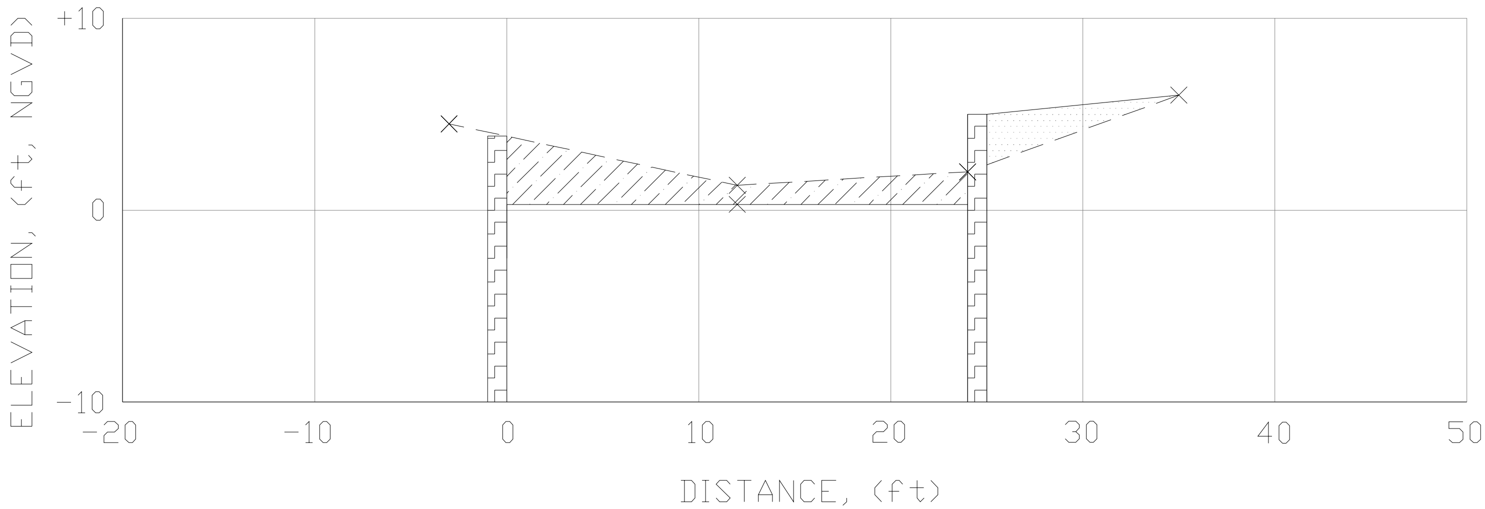
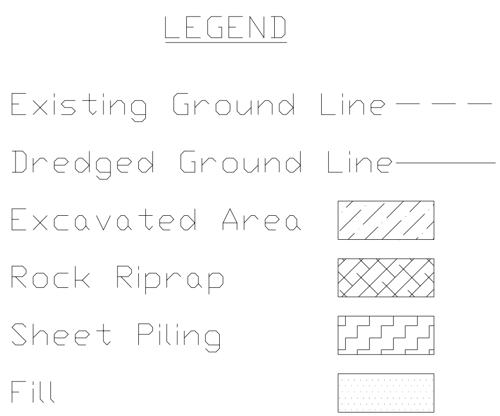
CROSS SECTION 5+25

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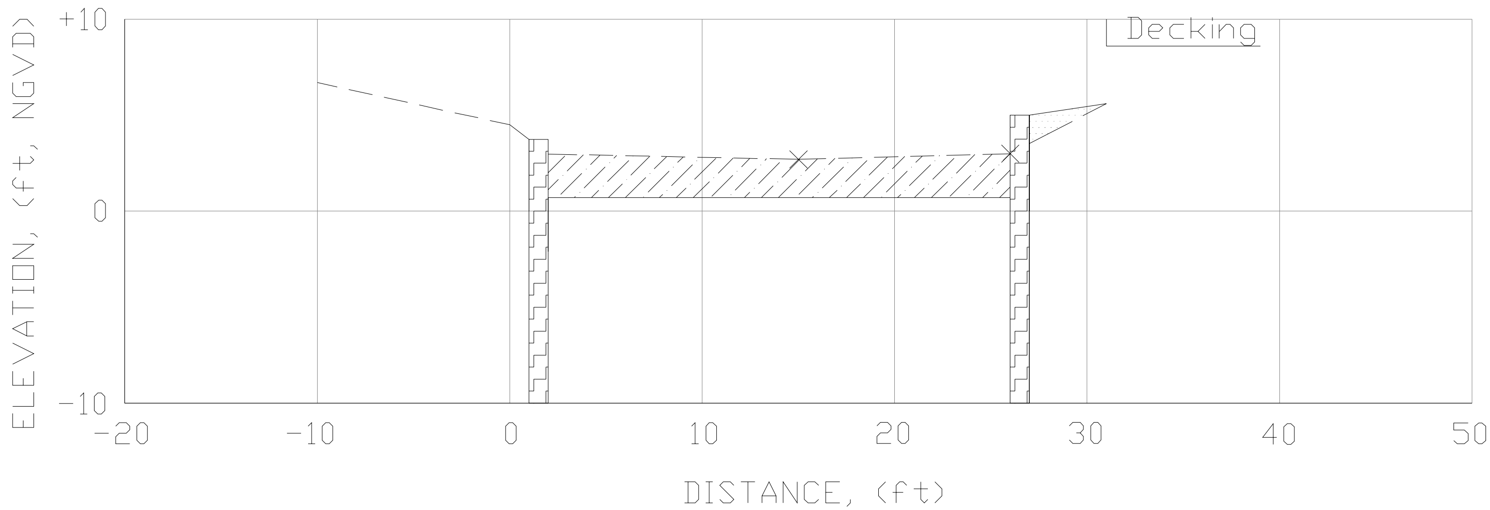
# CROSS SECTION 5+90





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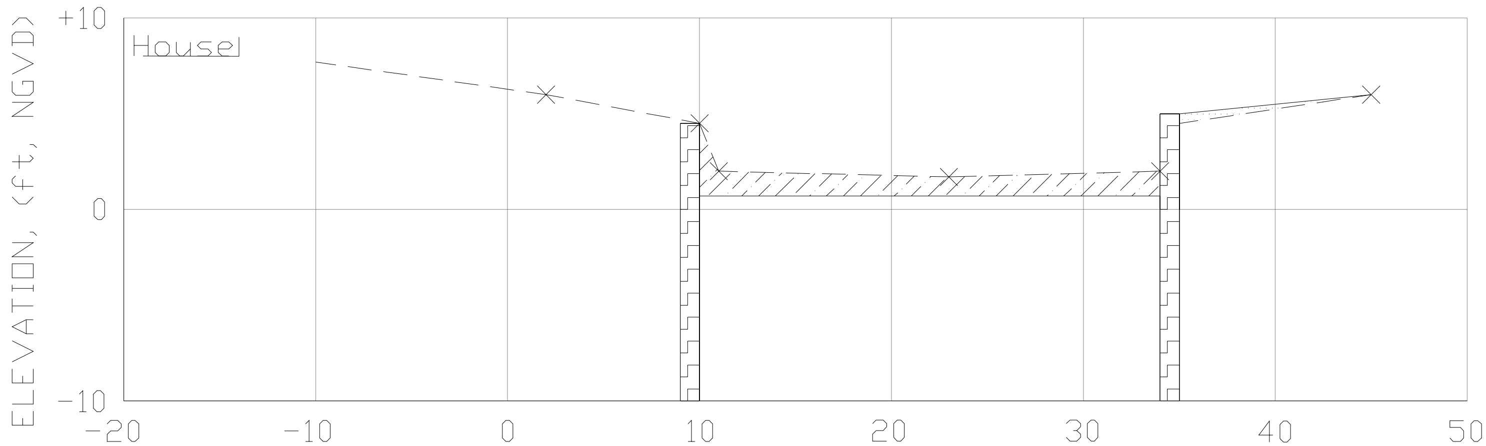
# CROSS SECTION 6+35




LEGEND

Existing Ground Line — — —

Dredged Ground Line ———

Excavated Area

Rock Riprap

Sheet Piling

Fill

PROPOSED ALTERNATIVE
EEL CREEK STABILIZATION PROJECT
HAMPTON, NH

U.S. DEPARTMENT OF AGRICULTURE
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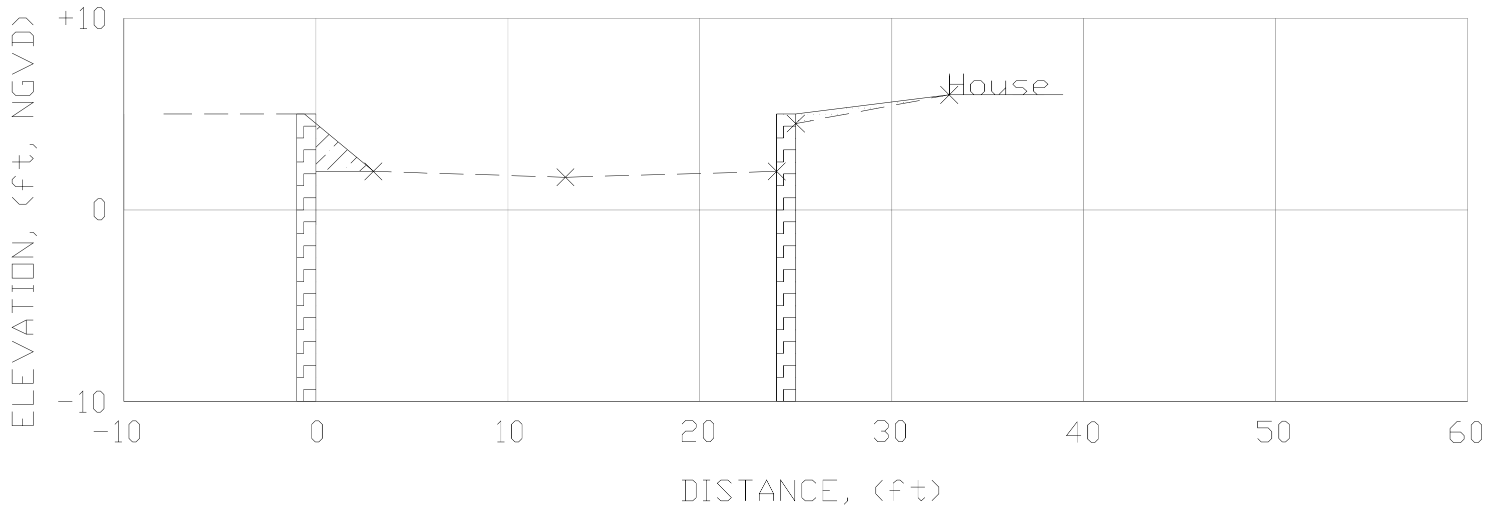
## CROSS SECTION 7+40




### LEGEND

- Existing Ground Line — — —
- Dredged Ground Line ———
- Excavated Area
- Rock Riprap
- Sheet Piling
- Fill

PROPOSED ALTERNATIVE
EEL CREEK STABILIZATION PROJECT
HAMPTON, NH

U.S. DEPARTMENT OF AGRICULTURE
NATURAL RESOURCES CONSERVATION SERVICE

| | Date | |
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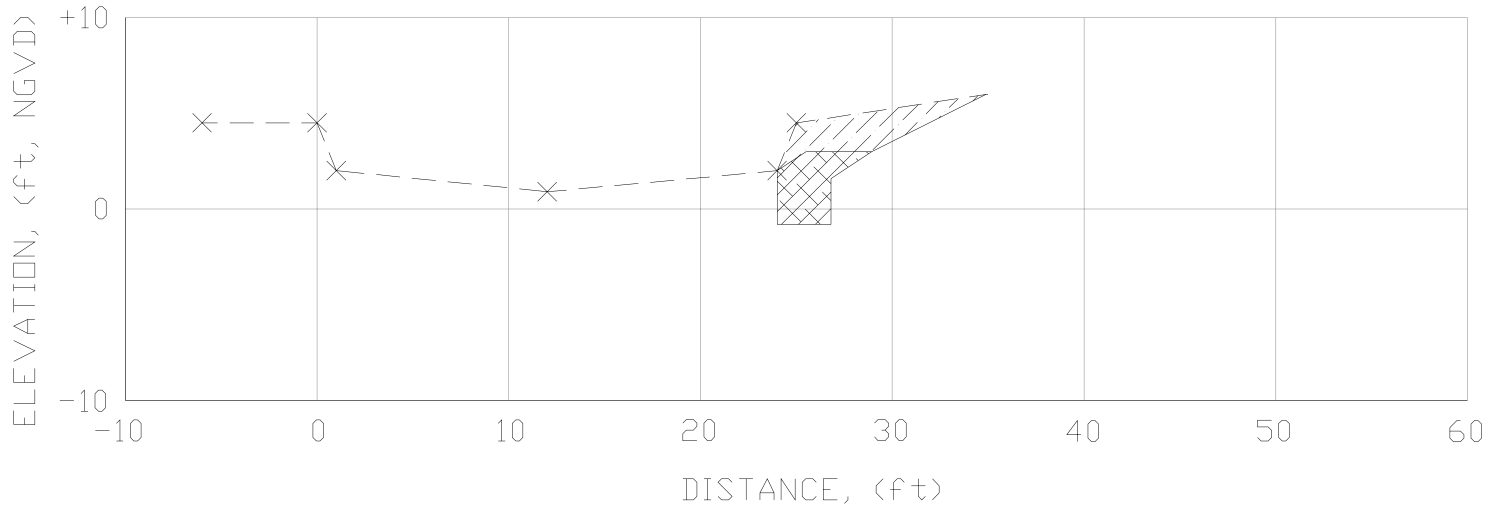
# CROSS SECTION 8+50




LEGEND

Existing Ground Line — — —

Dredged Ground Line ———

Excavated Area

Rock Riprap

Sheet Piling

Fill

PROPOSED ALTERNATIVE
EEL CREEK STABILIZATION PROJECT
HAMPTON, NH

U.S. DEPARTMENT OF AGRICULTURE
NATURAL RESOURCES CONSERVATION SERVICE

| | Date | |
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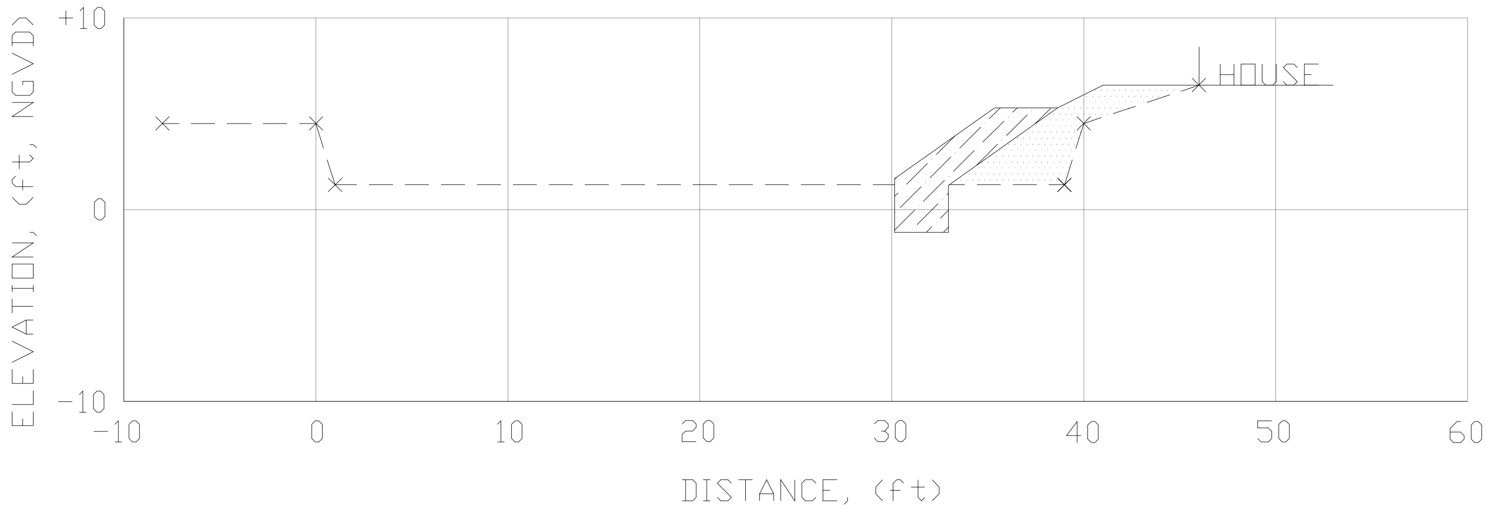
# CROSS SECTION 10+60




LEGEND

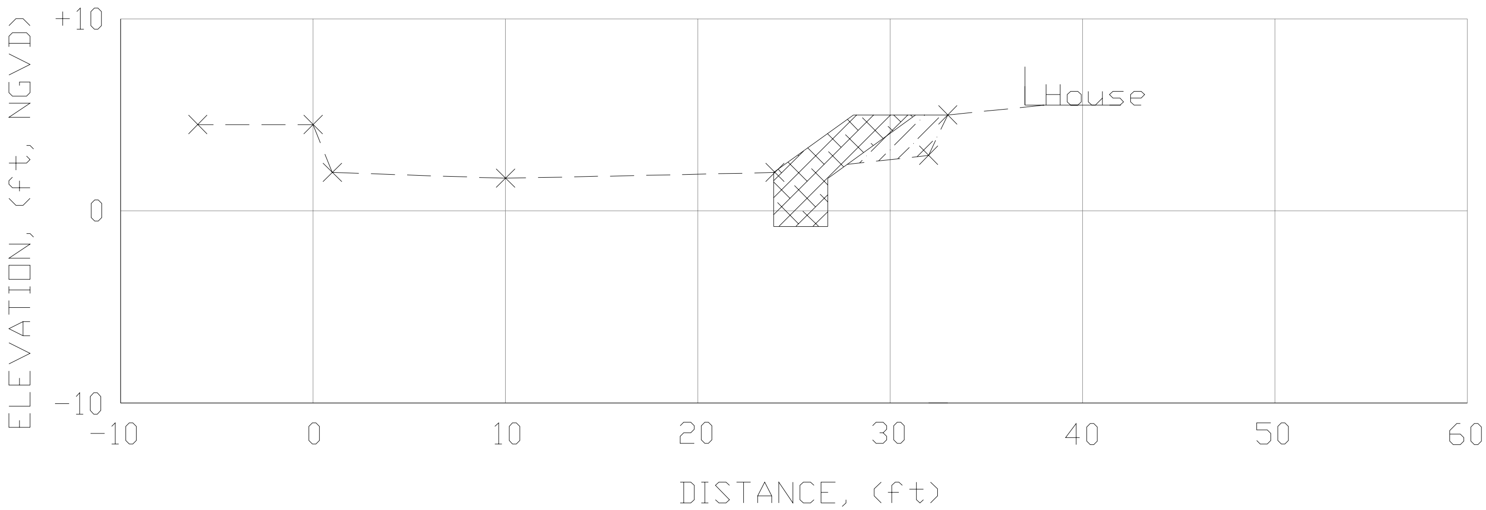
Existing Ground Line — — —
Dredged Ground Line ———
Excavated Area
Rock Riprap
Sheet Piling
Fill

PROPOSED ALTERNATIVE
EEL CREEK STABILIZATION PROJECT
HAMPTON, NH

U.S. DEPARTMENT OF AGRICULTURE
NATURAL RESOURCES CONSERVATION SERVICE

| Date | Approved by |
|---|---|
| Designed | Title |
| Drawn | |
| Traced | Title |
| Checked | Sheet of / of / Drawing No. |

## CROSS SECTION 11+10




### LEGEND

Existing Ground Line — — —

Dredged Ground Line ———

Excavated Area

Rock Riprap

Sheet Piling

Fill

PROPOSED ALTERNATIVE
EEL CREEK STABILIZATION PROJECT
HAMPTON, NH

U.S. DEPARTMENT OF AGRICULTURE
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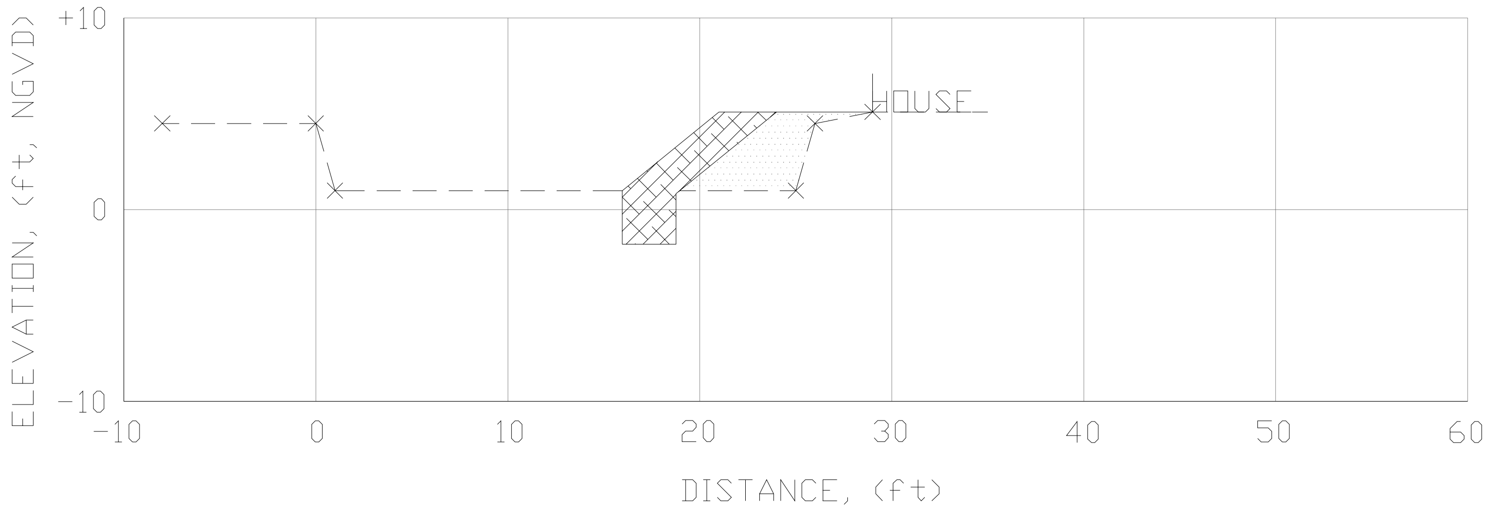
# CROSS SECTION 11+50




<u>LEGEND</u>

Existing Ground Line

Dredged Ground Line

Excavated Area

Rock Riprap

Sheet Piling

Fill

PROPOSED ALTERNATIVE
EEL CREEK STABILIZATION PROJECT
HAMPTON, NH

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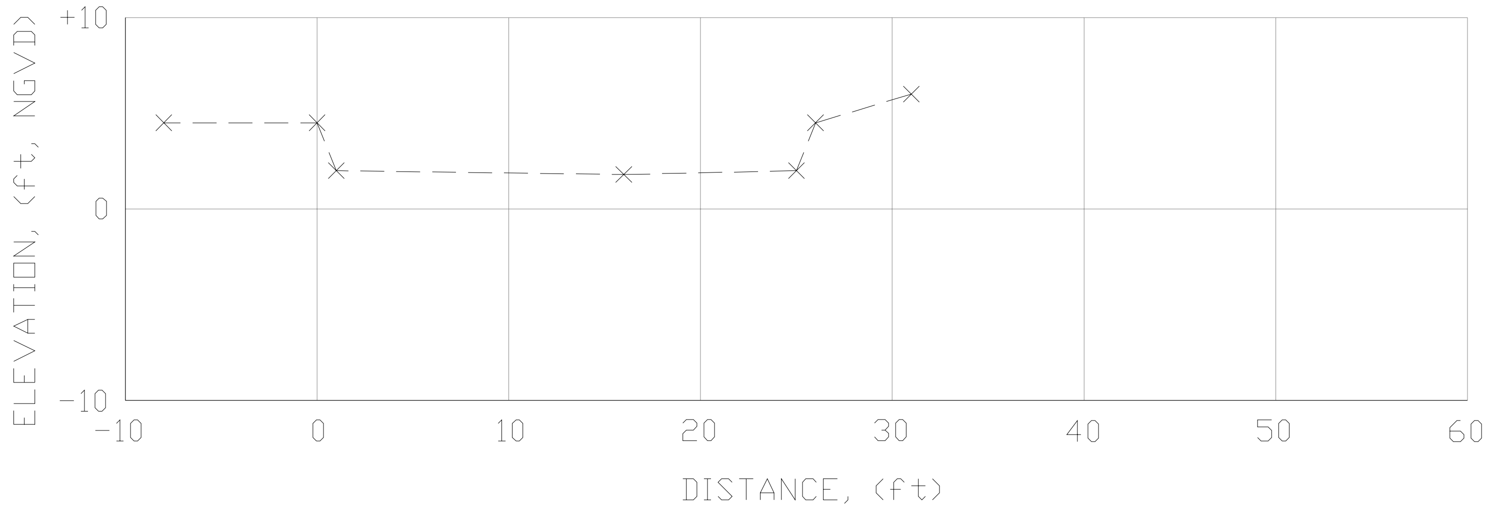
# CROSS SECTION 12+60




LEGEND

Existing Ground Line — — —

Dredged Ground Line ———

Excavated Area

Rock Riprap

Sheet Piling

Fill

PROPOSED ALTERNATIVE
EEL CREEK STABILIZATION PROJECT
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| Designed | | Approved by |
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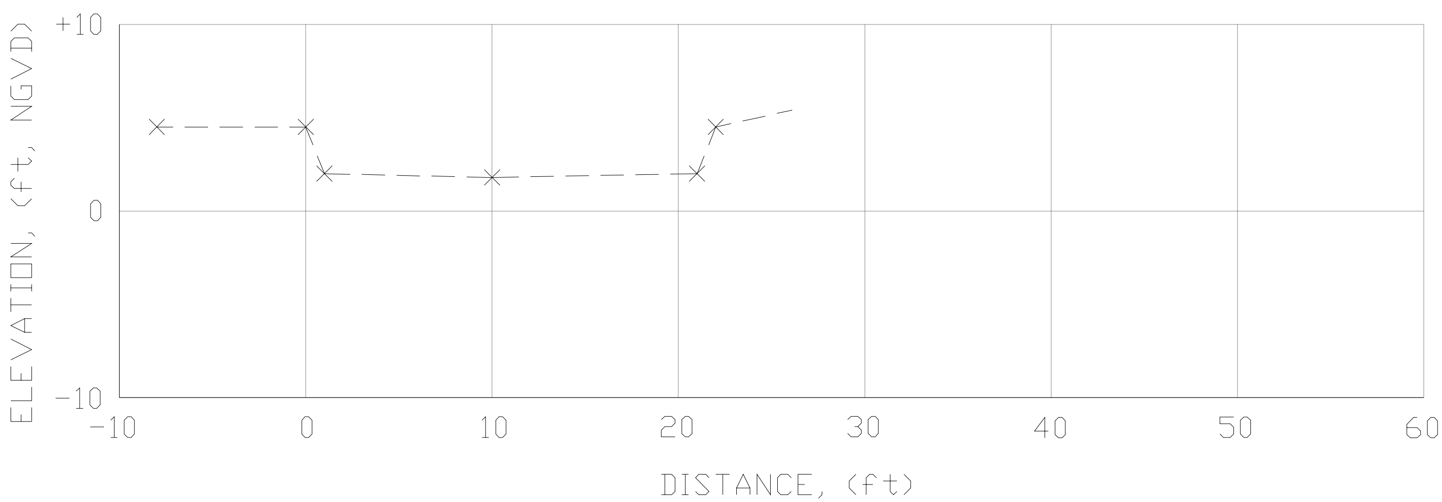
| Traced | | Title |
| Checked | | Sheet of / Drawing No. |

# CROSS SECTION 13+30




LEGEND

Existing Ground Line — — —

Dredged Ground Line ———

Excavated Area

Rock Riprap

Sheet Piling

Fill

PROPOSED ALTERNATIVE
EEL CREEK STABILIZATION PROJECT
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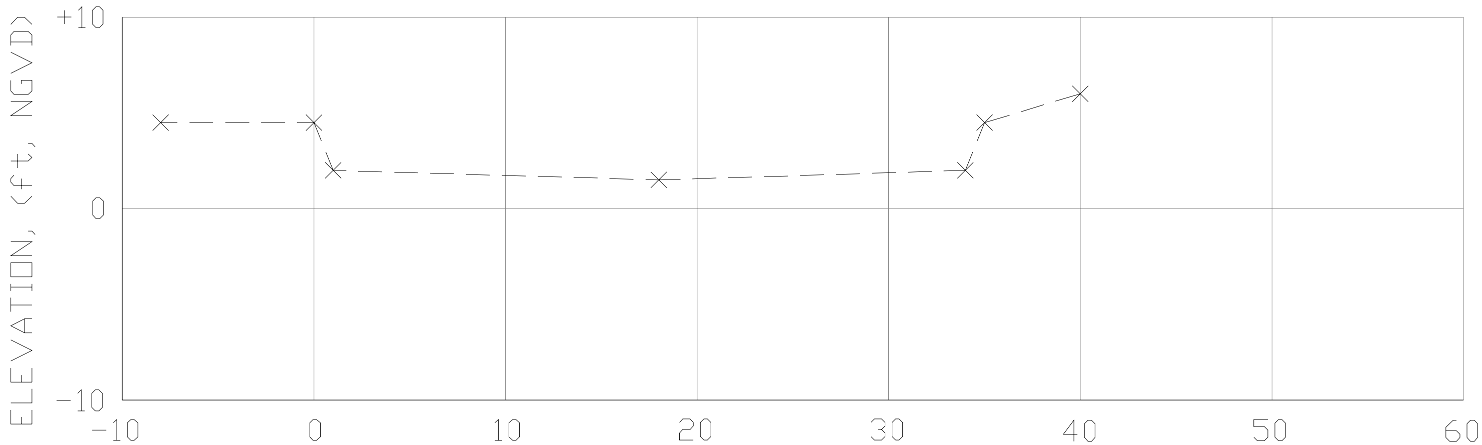
CROSS SECTION 14+50




LEGEND

Existing Ground Line — — —

Dredged Ground Line ———

Excavated Area

Rock Riprap

Sheet Piling

Fill

PROPOSED ALTERNATIVE
EEL CREEK STABILIZATION PROJECT
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| Designed | | Approved by |
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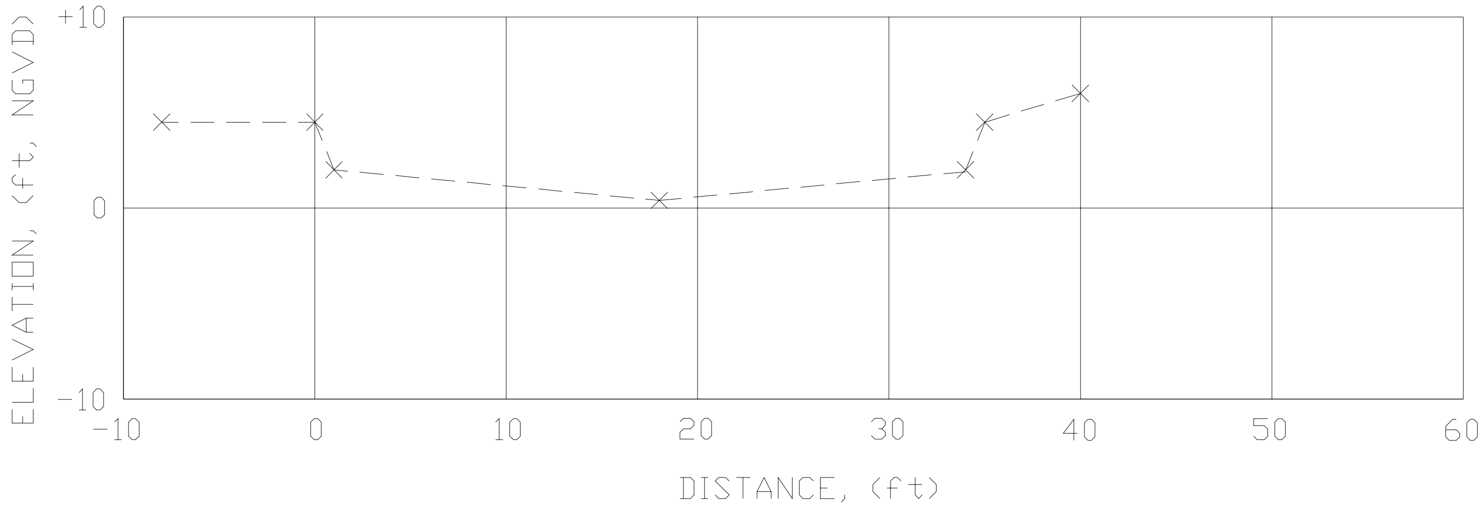
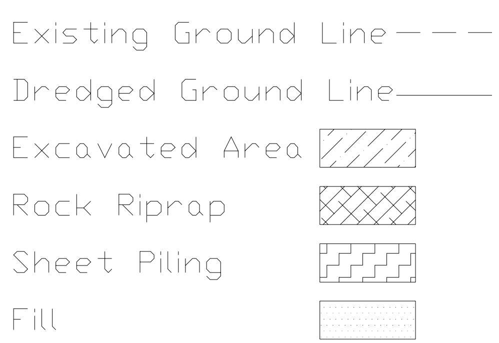
# CROSS SECTION 15+40




LEGEND

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| | Date | |
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| Designed | | Approved by |
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| Checked | | Sheet of / of — Drawing No. |